PARAMETRIC STUDIES ON STABILITY OF RESHAPED BERM BREAKWATER WITH CONCRETE CUBES AS ARMOR UNIT

Thesis

Submitted in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

By

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JULY 2014

D E C L A R A T I O N

I hereby *declare* that the Research Thesis entitled "**Parametric Studies on Stability of Reshaped Berm Breakwater with Concrete Cubes as Armor Unit"** which is being submitted to the **National Institute of Technology Karnataka, Surathkal** in partial fulfillment of the requirements for the award of the Degree of **Doctor of Philosophy** in **Civil Engineering** is a *bonafide report of the research work carried out by me.* The material contained in this Research Thesis has not been submitted to any University or Institution for the award of any degree.

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Dedicated to

My Teachers

My Family

ACKNOWLEDGEMENTS

With deep sense of gratitude, I express my heartfelt thanks to two of the imminent Professors from the Department of Applied Mechanics and Hydraulics, NITK, Surathkal, Prof. Subba Rao and Prof. Kiran G. Shirlal, for doing a marvelous job at supervising my research work. Their logical and tactical suggestions have been very valuable and encouraging during the development of this research. I acknowledge, the time spent in technical discussions with both my supervising professors in regards to the completion of this research, as immensely interesting and profoundly knowledge enhancing, to which I am greatly indebted to. Their moral support and critical guidance have been priceless which has given me an invaluable opportunity to publish my research work in many international/national journals/conferences which is a matter of great pride and satisfaction to me. The easily approachable nature and ever helpful attitude of Prof. Subba Rao and Prof. Kiran G. Shirlal will always be admired, valued and cherished.

I thank the former Director of NITK, Surathkal, Prof. Sandeep Sancheti, and present Director, Prof. Sawapan Bhattacharya, for granting me the permission to use the institutional infrastructure facilities, without which this research work would have been impossible.

I am grateful to Research Progress Committee members, Prof. M. S. Bhat and Prof. Katta Venkataramana, for their critical evaluation and useful suggestions during the progress of the work.

I am greatly indebted to Prof. M. K. Nagaraj, the former Head of the Department of Applied Mechanics and Hydraulics, NITK, Surathkal, and Prof. Subba Rao, the present Head of the Department, for granting me the permission to use the departmental computing and laboratory facilities available for necessary research work to the maximum extent which was very vital for the completion of the computational aspects relevant to this research.

I sincerely acknowledge the help and support rendered by all the Professors, Associate Professors and Assistant Professors of Department of Applied Mechanics & Hydraulics.

I gratefully acknowledge the support and all help rendered by the Post-Graduate students Sri Vishwanath N., Sri Sharath Kumar, Sri Mithun Shetty B. R., Smt. Geetha Kuntoji, Sri Danny Thomas and Sri Sainath Vaidya during the research work.

I take this opportunity to thank Late Prof. Alf Torum, NTNU, Norway and Dr. Balakrishna Rao, MIT, Manipal for their valuable suggestions during the research and all my friends for their support, suggestions and encouragement during my research.

Thanks due to Mr. Jagadish, Foreman, and his supporting staff, Mr. Ananda Devadiga, Mr. Gopalakrishna, Mr. Padmanabha Achar and Mr. Niranjan, for fabricating the moulds and casting concrete cubes is gratefully acknowledged.

I thank Mr. Seetharam, Draughtsman, for his help in drawing neat sketches and Mr. Balakrishna, Literary Assistant, is always remembered for his help in solving computer snags which cropped up during computations.

I express heart felt gratitude to authors of all those research publications which have been referred in this thesis.

Finally, I wish to express gratitude, love and affection to my beloved family members, father Sri. H. R. Janardhana, mother Smt. H. K. Mythily and sister Smt. Shubha J. and all my friends for their encouragement, moral support and all their big and small sacrifices on the road to the completion of my research.

Prashanth J.

ABSTRACT

The breakwater construction in deeper waters requires heavier armor units due to larger wave loads. Such large stones are uneconomical to quarry or transport or may not be available nearby. Another problem is uncertainty in the design conditions resulting in breakwater damage due to increased wave loads. The structural stability and economy in construction of breakwater are the need of hour.

Under these circumstances, berm breakwaters can be a solution. For an economical solution, the quarry yield may be judiciously used and berm breakwater may be constructed with small size armor units. The present research work involves a detailed experimental study of influence of various sea states and structural parameters on the stability of statically stable reshaped berm breakwater made of concrete cubes as primary armor.

Initially, a 0.70 m high of 1:30 scale model of conventional breakwater of 1V:1.5H slope and trapezoidal cross section is constructed on the flume bed with concrete cubes of weight 106 g as primary armor. This is designed for a non-breaking wave of height 0.10 m. This model is tested for armor stability with regular waves of heights 0.10 m to 0.16 m and periods 1.6 s to 2.6 s in water depths of 0.30 m, 0.35 m and 0.40 m.

In the second phase, a 1V:1.5H sloped and 0.70 m high berm breakwater with varying size of concrete armor cubes, berm widths, thickness of primary layer is tested for stability with same test conditions.

Based on the study of conventional and reshaped berm breakwater model the following conclusions are drawn. Damage level (S) was found increasing with the increase in stability number (N_s) in conventional breakwater. In conventional breakwater damages were in the range of 4.62 to 5.69 (intermediate), 9.75 to 11.46 (failure) and 9.46 to 10.22 (failure) in the depths of 0.3m, 0.35m and 0.4m respectively. Considering the complete ranges of H_o/gT^2 and d/gT^2, the maximum relative run-up R_u/H_o and relative run-down R_d/H_o were respectively 1.2 and 1.25. The stability of the berm breakwater is largely influenced by the storm duration. It was observed that relative berm position (h_b/d) has a greater influence on berm recession than wave run-up and run-down. As relative berm position (h_b/d) parameter increases from 1.00 to 1.50, the berm recession decreased by up to 77% while the wave run-up and run-down decreases by 7% and 14% respectively. The surface elevation of the water in front of the berm influences the recession and eroded area of the berm. Some of the available equations for berm recession, wave run-up over estimated the values for the considered conditions. The damage is reduced by about 47% in the present model when compared to stone armored berm breakwater. The wave run-up and run-down are reduced by 34% and 49% compared to conventional cube armored breakwater respectively. The economic analysis showed that the cube armored berm breakwater is about 8% and 4% economical than the conventional cube armored breakwater and stone armored berm breakwater for the same design conditions. The design equations for berm recession, wave run-up and wave run-down are derived.

Finally, it was found that 25% reduction in armor weight with 0.40 m berm width and 2 no. of primary armor layers is safe for the most of conditions considered during the study except for extreme waves of 0.16 m height and 1.6 s period. However, same breakwater with 3 armor layers was safe for the entire range of test conditions. In terms of safety as well as economy 25% reduction in armor weight with 0.40 m berm width and 2 no. of primary layer was cheaper compared to all other models studied.

Keywords: berm breakwater, concrete cube armor, armor weight, berm width, wave climate, water depth, wave run-up, wave run-down, berm recession

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NOMENCLATURE

A_e	=	Eroded area of sea-ward profile in cross-section
B	=	Berm width
D_{n50}	=	Nominal diameter of the breakwater armour unit
D_{15}	=	15% of the stones have a diameter less than D_{15}
D_{85}	=	85% of the stones have a diameter less than D_{85}
d	=	Water depth in front of structure
d_B	=	Water surface above or below the berm horizontal surface
f_g	=	Gradation factor, D_{85}/D_{15}
f_d	=	Depth factor
g	=	Gravitational acceleration
H	=	Wave height in front of the structure
H_s	=	Significant wave height
h_b	=	Berm height above sea bed
H_o	=	Deep water wave height
H_oT_o	=	Wave height- period stability number
K_D	=	Stability coefficient in Hudson formula
k_Δ	=	Armor layer coefficient
L_o	=	Deep water wave length
N	=	Number of waves
N_s	=	Stability number
n	=	Sample size, number of armor layers
P	=	Permeability coefficient
R_d	=	Wave run-down
R_u	=	Wave run-up
R_{ui}	=	Run-up level exceeded by '*i*' percent of the incident wave
R_e	=	Reynolds number
R_{ec}	=	Recession of the reshaped berm
R_c	=	Crest height of structure

s	=	Wave steepness
T	=	Wave period
T_z	=	Zero up-crossing wave period
t	=	t-distribution values from statistical table
W_{50}	=	50% value of mass distribution curve of armor units
W_{15}	=	15% value of mass distribution curve of armor units
W_{85}	=	85% value of mass distribution curve of armor units
t_a	=	Thickness of armor layer
x_c	=	Crest width
x	=	Variable
$\bar{x}$	=	Sample mean
σ	=	Standard deviation
Δ	=	Relative mass density, $(\rho_a - \rho_w)\rho_w$
α	=	Angle of seaward slope of structure
α_{eq}	=	Equivalent slope angle of berm breakwater structure
β	=	Angle of incidence of wave direction
β_o	=	Intercept - regression coefficients
β_1	=	Slope - regression coefficients
γ_r	=	Reduction factor for influence of surface roughness
γ_b	=	Reduction factor for influence of a berm
γ_h	=	Reduction factor for influence of shallow-water conditions
γ_β	=	Reduction factor for influence of angle of incidence β of the waves
γ_a	=	Specific weight of armor unit
ρ_a	=	Mass density of armor unit
ρ_w	=	Mass density of water
ξ_{om}	=	Deep water surf-similarity parameter, $\mathrm{Tan}\alpha/\sqrt{(H_o/L_o)}$
ξ_{eq}	=	Breaking wave surf similarity parameter of an equivalent slope
S	=	Damage level
R_{ec}/D_{n50}	=	Dimensionless berm recession

R_{ec}/B	=	Relative berm recession
$H/\Delta D_{n50}$	=	Hudson's stability number, N_s
H_o/gT^2	=	Deepwater wave steepness, s
B/d	=	Relative berm width
h_b/d	=	Relative berm position
$\tan\alpha/(s)^{-0.5}$	=	Surf similarity parameter, ξ
d/gT^2	=	Relative water depth
R_u/H_o	=	Relative run-up
R_d/H_o	=	Relative run-down
$\cot\alpha$	=	Cotangent of breakwater slope
R^2	=	Co-efficient of determination

CHAPTER 1

INTRODUCTION

1.1 GENERAL

The requirement of any port, harbor or marina is a sheltered area free from the sea waves. In the coastal areas where natural protection from waves is not available, the development of harbor requires an artificial protection for the creation of calm areas. For harbors, where perfect tranquility conditions are required, large structures such as rubble mound breakwaters or vertical wall breakwaters are used. Most of the breakwaters are used to create tranquil conditions in the lagoon and at the entrance channel of ports, for maneuvering of ships and port operations. Many a times breakwaters are also used as berthing structures along with protecting the harbor area. Sometimes they are used to protect beaches from erosion due to the destructive wave forces (Verhagen et al. 2009).

The selection of the type of breakwater will be primarily based on the function of the breakwater, wave climate of that area, depth of water, availability of construction materials and local labor, geotechnical characteristics of sea bed, environmental concerns and available contractor potential. Although there are developments in construction technology, the rubble mound structures remain the most commonly used among all types of breakwaters worldwide for more than a century now to protect harbor basins against the wave forces.

Rubble mound breakwater is typically a three layered breakwater of trapezoidal shape. It is used in relatively shallow water wave conditions only. However, in 1970s the need for bigger ships and harbors has resulted in construction of breakwaters in deep water where experience was seldom available. This development resulted in the failure of several huge rubble mound breakwaters. Further, in order to withstand the high waves, breakwaters were built with large heavy rocks. But this posed a problem since the maximum rock size was limited and in some parts of the world no large size rock or good quality rock was available. Additionally, from the economic point of view, breakwaters represent a significant portion of a capital investment in the development of a port, and would require regular maintenance to retain their effectiveness. Hence, an analysis of the total cost over the lifetime of the structure was also essential. This is another reason that makes it necessary to search for better solutions and develop economical and safe structures to serve a particular purpose.

1.2 BREAKWATERS AND ITS ARMOR UNITS

1.2.1 History of Breakwaters

The construction of structures on sea dates back to around 5000 BC with the building of sea link between then India and Srilanka as quoted in the great Indian epic Ramayana (Bala 2013). The development of breakwater construction is closely related to the development of ports around the world over the centuries. The first breakwaters built can be linked to the ancient Egyptian, Phoenician, Greek and Roman cultures. The structures were simple mound structures, composed of locally found rock. As early as 2000 BC, mention was made of a stone masonry breakwater in Alexandria, Egypt (Takahashi 1996). The Roman emperor Trajan (AD 53-117) initiated the construction of a rubble mound breakwater in Civitavecchia, which still exists today (Fig. 1.1) (D'Angremond and van Roode 2004).

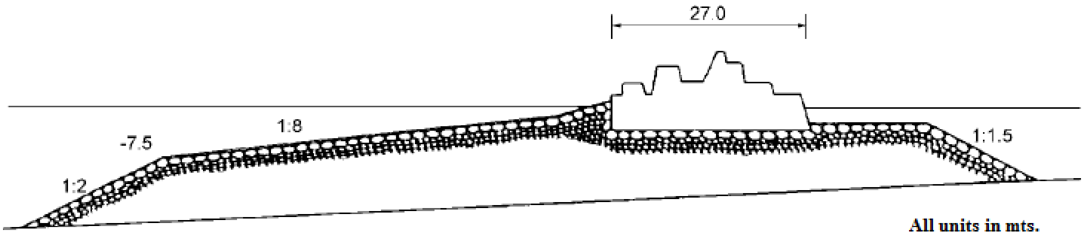
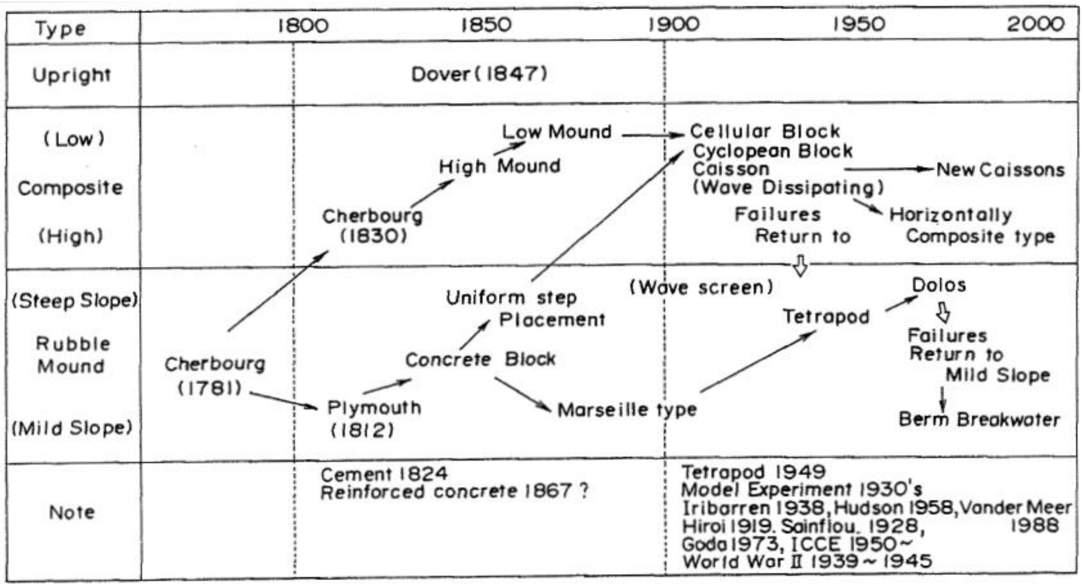


Fig. 1.1 Rubble mound breakwater at Citavecchia (D'Angremond and van Roode 2004)

The standards for design and construction of a breakwater remained those developed primarily by the Romans, later a great leap in technology was achieved through the development of mechanical equipment and hydraulic sciences including maritime hydraulics (Franco 1996). De Cessart undertook rather complex work of a breakwater construction in Cherbourg harbor in comparatively deep waters. The Plymouth breakwater started in 1811 showed a remarkable similarity of profile with the Cherbourg breakwater (Bruun 1985). The characteristic feature of these breakwaters was that they were periodically and partially damaged due to storms. The stable profile of these breakwaters closely resemble the ‘S’ profile adopted for breakwaters later. The vertical wall breakwater at Dover was constructed in1847 which worked very well compared to breakwaters built in Cherbourg and Plymouth. The Cherbourg, Plymouth and Dover breakwaters are considered to be the pioneers of modern-day breakwater structures (Takahashi 1996). Table 1.1 shows the historical development of breakwaters along with the development of armor units (Takahashi 1996).

Table 1.1 Summary of historical development of breakwater (Takahashi 1996)



1.2.2 Types of breakwaters

The breakwaters are mainly classified as:

1. Rubble mound or heap breakwaters.
2. Upright or vertical wall breakwaters.
3. Mound with superstructure or composite breakwaters.
4. Special types of breakwaters.

<u>1. Rubble mound breakwaters:</u> It is a heterogeneous assemblage of natural rubble or undressed stone blocks or in many cases by artificial blocks. It is of trapezoidal shape with stones or blocks being deposited on its slope without any regard to bond or bedding. This is the simplest type and is constructed by tipping or dumping of rubble stones into the sea till the heap or mound emerges out of the water, the mound being consolidated and its side slopes regulated by the action of the waves.

Rubble mound breakwaters are suitable for all types of foundations. They can be constructed up to 50 m depth economically and can be repaired easily and periodically. Even though initial and maintenance cost is high, construction does not necessitate skilled labor or specialized equipments. But the material required might be of enormous quantity and heavy at locations having large tidal ranges, high waves and deeper depths. Further, the rubble mound breakwaters can be classified based on its variation in structural configuration as Multi layer rubble mound breakwater, submerged breakwaters, berm breakwaters, reef breakwaters and tandem breakwaters (Refer Fig. 1.2).

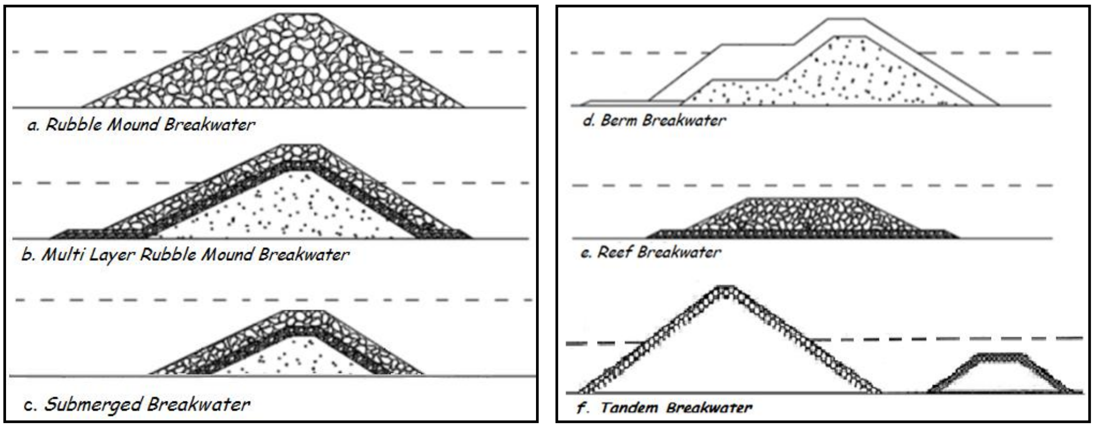


Fig. 1.2 Different kinds of rubble mound breakwaters

Multi layer rubble mound breakwaters are made of three layers namely primary layer, secondary layer and a core. The primary layer is directly exposed to waves and also acts a protective layer for secondary and core layers. A submerged breakwater is similar to multi layer breakwater with only its crest at or below sea level. A breakwater with a horizontal berm at some elevation is called a berm breakwater. The berm breakwater may be reshaping type or non-reshaping type. The traditional rubble mound is a reef structure constructed as a heap of bulky stones laid at some stable slope to resist the wave action. A reef breakwater is a low-crested group of stones without a filler layer or core. Reef breakwater is allowed to reshape under wave attack. The concept of rubble mound breakwater and submerged reef breakwater, operating together as a single unit, is called the tandem breakwater. The submerged reef reflects and dissipates the wave energy by inducing wave breaking, thereby, reducing impact of waves on the main breakwater (Shirlal and Rao 2003). A detailed study on rubble mound breakwaters in particular berm breakwater is given in later chapters.

<u>2. Upright or vertical wall breakwaters:</u> These breakwaters are of types such as huge concrete blocks, gravity walls, concrete caissons, rock filled timber cribs and concrete or steel sheet pile walls as indicated in Fig. 1.3. Vertical wall structures are used as breakwaters, seawalls, and bulkheads in harbors. The main purpose of a vertical breakwater is to reflect waves, while that for the rubble mound breakwater is to break them and dissipate wave energy. They are used in relatively moderate wave conditions but when a critical load value is exceeded, these monolithic breakwaters lose their stability at once. This catastrophic failure due to loss of stability is one of the major disadvantages for this type of breakwaters.

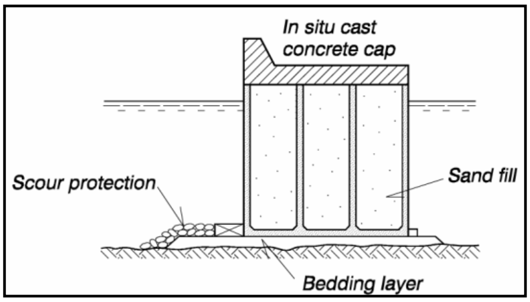


Fig. 1.3 Conventional caisson breakwaters with vertical front

<u>3. Mound with superstructure or composite breakwaters:</u> Composite breakwaters are combination of rubble mound and vertical wall breakwaters. These are used in locations where either the depth of water is large or there is a large tidal range and in such situations, the quantity of rubble stone required to construct a breakwater to the full height would be too large. In such conditions, a composite breakwater is constructed as a structure with rubble mound base and a super structure of vertical wall as shown in Fig. 1.4. These breakwaters tend to fail when the waves break near the mound and then slam against the vertical wall, which damages the structure.

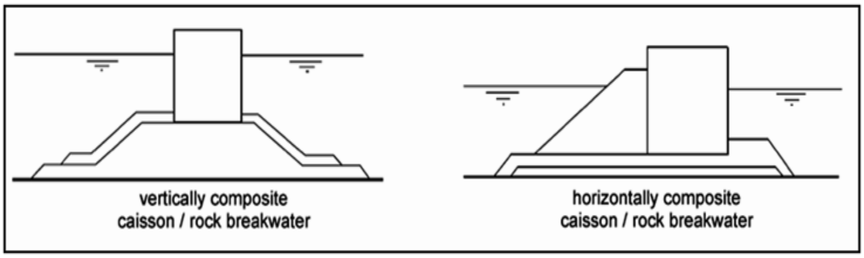


Fig. 1.4 Composite breakwaters

4. Special type of breakwaters: These breakwaters are for specific purposes with special features and are not commonly used. Depending on the site condition these are designed. Special type of breakwaters can be divided into two types. One is the non-gravity type breakwaters such as pile type, floating, pneumatic etc. The other is gravity type which is the conventional breakwater with special features conceived to improve the functioning and stability of breakwater. Some special breakwaters are as follows:

- Curtain wall breakwater – commonly used as secondary breakwater to protect small craft harbors.
- Sheet pile walls – are used to break relatively small waves.
- Horizontal plate breakwater – can reflect and break waves and are supported by a steel jacket.
- Floating breakwater – Used in deep waters especially in places where the ground soil is poor for foundation
- The pneumatic breakwater – breaks the wave due to water current induced by air bubbles.

1.2.3 Armor units

An armor unit is a massive and bulky stone or concrete unit placed as a primary layer material in rubble mound breakwaters to resist the wave attack. The armor units are selected to fit specified geometric characteristics and density. The units require individual placement on the structure since units are very large in size. The choice of type of unit depends mainly on availability of quarry and the mass of unit required. The armor units are mainly classified into two categories namely natural armor units and artificial armor units. The quarried stone is a natural unit and concrete unit is an artificial armor unit.

In the early days when breakwaters were built in relative shallow waters, natural stones were used as armor units since the required size and weight were easily available from quarries nearby. As the construction started moving into deeper waters, the wave load on armor increased requiring heavier armor units. It became difficult to procure such heavy weight stones or even to transport them to the site. Sometimes such heavy weight armor stones were not available in the nearby quarries. This posed a problem as breakwaters constitute the main part of the port from economic point of view. In order to overcome this disadvantage and with the advancement in the field of concrete technology new type of armors of various shapes and sizes were developed, and these were known as artificial armor units. These units can be manufactured/cast at sites as per the requirement.

The artificial armor units are found to be more advantageous than natural armor units. The artificial armor units can be of desired shape, size and weight which are relatively lighter compared to natural armors and can be cast in-situ. Also, they can attain better interlocking capability, improved hydraulic and structural stability compared to natural units. Their optimum interlocking capability is balanced between hydraulic stability and the ease of fabrication.

The usage of concrete blocks dates back to early 19th century. The historical port of Algiers which was constructed in 16th century was protected by rubble mound breakwater made of stones which was getting continuously damaged. In 1833, a French engineer, Poirel, carried out reinforcement work using 2 to 3 m^3 stones, but the stones ended up being unstable. The breakwater was later successfully reinforced using 15 m^3 rectangular concrete blocks (Takahashi 1996). Fig. 1.5 shows the north breakwater of Algiers port armored with 15 m^3 concrete blocks with a slope of 1:1. Further, many breakwaters were built in various ports in Algeris *viz.* Algeris, Oran, Philippeville etc. As quoted by Takahashi (1996) these breakwaters also suffered damage due to steep slope, insufficient weight of blocks, insufficient depth of armor layer and random placement of blocks.

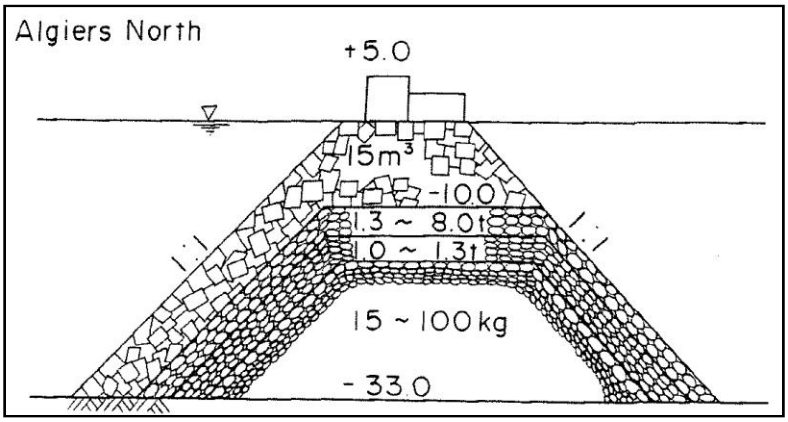


Fig. 1.5 Algeris north breakwater (Takahashi 1996)

A fairly new concept was adopted for the breakwater built in Marseille to overcome the failure in previous breakwaters. The breakwater was made strong with some special features. The inner core was included with lighter stones covered by heavier stones. The primary layer of concrete block was extended to sufficient depth and the slope above sea level was kept gentle (1:3). The lower slope below sea level was kept steep (3:4) compared to upper slope and the armor blocks in this region were placed carefully. Fig. 1.6 shows the Marseille breakwater constructed with the above features. The breakwaters constructed later followed this concept and they were called as Marseille type.

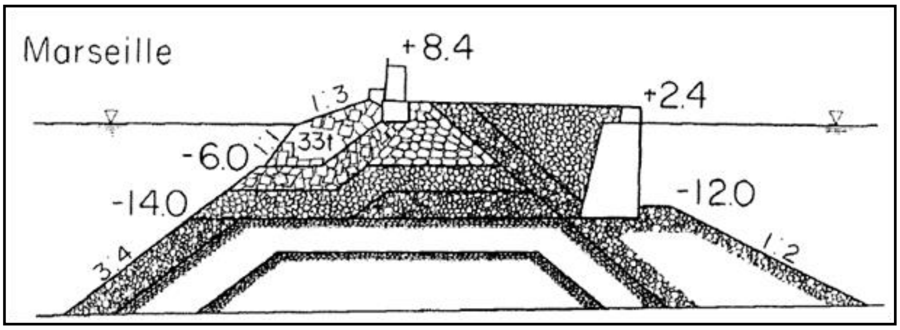


Fig. 1.6 Marseille breakwater (Takahashi 1996)

The concrete block unit was very massive and heavy which produced large cross sections for mild slope above sea level. This made construction difficult and uneconomical. These disadvantages were overcome with the advent of interlocking concrete blocks in 1949 by P. Danel. 'Tetrapods' thus developed marked the start of a long series of similar blocks. Danel (1953) summarized the characteristics of interlocking blocks as:

1. Porosity: Impermeable layers caused internal pressure in the mound thus disrupting the structure. Permeable layer tend to dissipate the wave energy thus reducing internal pressure, wave run-up and also wave run-down.

2. Roughness: The friction between water layer and armor layer helps in reducing the wave energy. Also, friction between the blocks keeps them in their position reducing the damage. The above two can be achieved by increasing roughness of the blocks which is possible by providing interlocking between them with projecting legs.

3. Resistance: The block must be strong enough to withstand breakage. If a very long leg/projection is provided there will be chances of breaking and with a short projection there will be no adequate resistance and interlocking developed.

In order to achieve all the above characteristics an optimum length of projection was needed. Thereafter many armor units with different shapes and a number of projections were developed. The emphasis was mainly on the interlocking of the units. The slender armor units 'Dolos' developed by Merrifield in South Africa in 1968 probably represents the peak of this concept and has a very high degree of interlocking. The failure of Sines breakwater which was armored with 42T Dolos unit was a major setback for this unit. This failure led to the development of stronger complex unit, still with higher hydraulic stability. The result of such development is Accropode developed by Sogreah in 1980 which brought an end to the rapid development of interlocking units.

In late 1960's a parallel development of completely different type of armor was in progress. The armor layer consisting of hollow blocks is placed uniformly in a single layer where each block is tied to its position by the neighboring blocks. This armor concept was not based on weight or interlocking but on friction between blocks, which provided an extremely high degree of hydraulic stability.

Many studies by investigators showed that voids played a very important part in the stability of armor layers. It was in these voids, created by units in a pack, that the greatest dissipation of wave energy takes place by relieving fluid pressure trying to move the blocks. For certain geometric shapes, the percentage of voids in the armor layer increased drastically as compared to conventional rectangular block armor layers. Based on this void concept many hollow blocks were developed like Cob, Shed etc.

Presently, the development of new types of armor block has reduced instead, now optimization of breakwater structure with a selected armor unit is being done. The selection of armor unit depends on many factors. Some of commonly used armor units are shown in Fig. 1.7

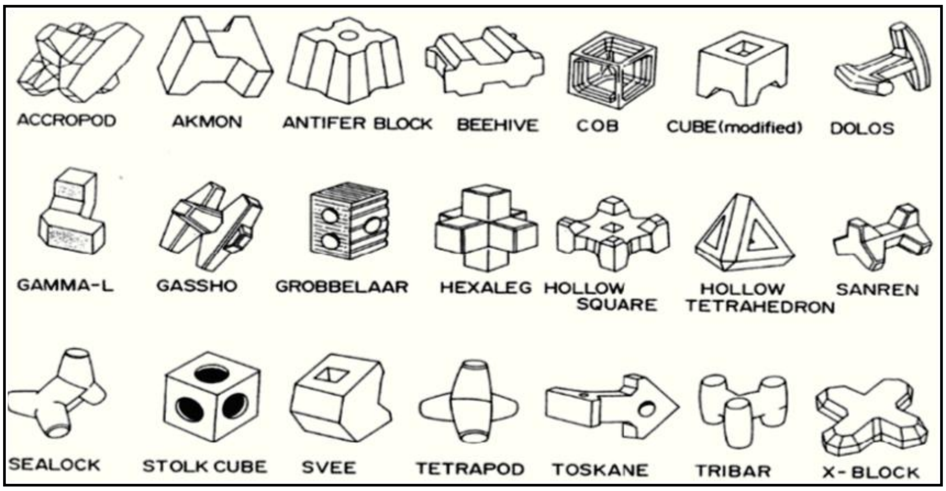


Fig. 1.7 Examples of Artificial Armor Units (Pilarczyk and Zeidler 1996)

1.2.4 Classification of artificial armor units

Bakker et al. (2003) classified the concrete blocks based on risk of progressive failure as:

Compact blocks: The armor unit's own weight is the main stabilizing factor and these units posses low average hydraulic stability. However, the structural stability is high and the variation in hydraulic stability is relatively low. Thus, the armor layer can be considered as a parallel system with a low risk of progressive failure.

Slender Blocks: In these blocks, interlocking is the main stability parameter and the units have large average hydraulic stability. However, the variation in hydraulic resistance is also relatively large and the structural stability is low. Therefore, slender blocks shall be considered as a series system with a large risk of progressive failure.

Further, breakwater armor units can be classified based on their shape as shown in Table 1.2.

Table 1.2 Classification of breakwater armor units by shape (Bakker et al. 2003)

Shape	**Armor Blocks**
Cubical	Cube, Antifer Cube, Modified Cube, Grobbelar, Cob, Shed
Double anchor	Dolos, Akmon, Toskane
Thetraeder	Tetrapod, Tetrahedron (solid, perforated, hollow), Tripod
Combined bars	2-D: Accropode, Gassho, Core-Loc 3-D: Hexapod, Hexaleg, A-Jack
L-shaped blocks	Bipod
Slab type (various shape)	Tribar, Trilong, N-Shaped Block, Hollow Square
Others	Stabit, Seabee

A more general classification based on shape, stability and placement pattern divides the most commonly used armor units in to 6 categories as shown in Table 1.3 below.

Table 1.3 Classification of armor units by shape, placement and stability factor (Bakker et al. 2003)

<table>
<tr><th rowspan="2">Placement pattern</th><th rowspan="2">No. of layers</th><th rowspan="2">Shape</th><th colspan="3">Stability factor</th></tr>
<tr><th>Own weight</th><th>Interlocking</th><th>Friction</th></tr>
<tr><td rowspan="5">Random</td><td rowspan="3">Double layer</td><td>Simple</td><td>(1) Cube, Antifer Cube, Modified Cube</td><td></td><td></td></tr>
<tr><td rowspan="2">Complex</td><td colspan="2">(2) Tetrapod, Akmon, Tribar, Tripod</td><td rowspan="2"></td></tr>
<tr><td></td><td>(3) Stabit, Dolos</td></tr>
<tr><td rowspan="2">Single layer</td><td>Simple</td><td>(5) Cube</td><td rowspan="2">(4) A-Jack Accropode, Core-Loc</td><td></td></tr>
<tr><td>Complex</td><td></td><td></td></tr>
<tr><td rowspan="2">Uniform</td><td rowspan="2">Single layer</td><td>Simple</td><td></td><td></td><td>(6) Seabee, Hollow cube, Diahitis</td></tr>
<tr><td>Complex</td><td></td><td></td><td>Cob, Shed</td></tr>
</table>

From the above discussions it is observed that the construction of breakwaters was mainly by trial and error. The failure of earlier breakwaters helped in bringing modifications in the design of breakwaters that were built later. The development of artificial armor units brought about larger implementation of rubble mound breakwater as a protective structure that too in greater depths. Even though the use of concrete units as armor was found to be more efficient than stones, the failure of the breakwater could not be avoided. The following section gives a brief account of failures of some major breakwaters.

1.3 FAILURE OF RUBBLE MOUND BREAKWATERS

If a structure fails to perform its designed function it is considered as failure of the structure. The rubble mound breakwaters are flexible structures in which the catastrophic failure is less. The earlier stone armored breakwaters failed gradually and partially showing visible signs of failure before they failed completely. But, with the use of artificial armor units which were shape specific and mainly depended on its interlocking property, the failure was very impulsive. These concrete units were slender and had protruding legs to ensure interlocking. In case of interlocking units during construction itself some units would develop cracks or get damaged. Under the wave attack they got further damaged due to rocking or movements. This caused the units to lose their interlocking ability and the armor unit could then be easily displaced by wave action. Also these broken units would crash against neighboring armor units, causing further breakage. This lead in a rapid unraveling of the armor layer, exposing the under layer and the core to direct wave action. These layers which were not being designed to resist direct action of waves got degraded very fast causing a catastrophic failure of the structure. Bruun and Kjelstrup (1983) have given some common criteria/reasons for the failure of rubble mound breakwaters which include,

1. 'Knock-out by plunging waves when

$$\xi_b = tan\alpha\sqrt{\frac{H_b}{L_o}} within\ the\ range\ 0.5 - 2.0 \text{ ---------------------------------- } (1.1)$$

2. Liftouts (by uprush - downrush) usually resulting from combined velocities in an arriving plunging wave.

3. Slides of the armor as a whole. This happens in case of steep slopes which are subjected to higher wave periods close to resonance.

4. Gradual breakdown or failure due to fatigue.

Some of the failures of breakwaters have been explained in following paragraphs. These breakwaters were built using both natural as well as artificial armor units. The damaged breakwater have been later rehabilitated by providing an additional structure or replacing the whole with a new structure depending on the damage to the existing structure.

The largest and most disastrous failure is that of Sines breakwater in Portugal, constructed with 42T Dolos, which failed in 1979 when, cyclonic waves ruptured the slender web portion of armor units resulting in loss of interlocking between them. This was not the only reason for the failure of this breakwater (Ziedler et al. 1992). Inadequate wave data collection, insufficient experimental studies and usage of inadequate armor in deepwater and structurally weak armor units were the other reasons which added up during the cyclonic storm resulting in the catastrophic failure of this breakwater.

The large port for export of LNG at Arzew El Djeded, Algeria was protected by a 2 km long main breakwater constructed in water depths of about 25 m. The breakwater was armored with 48T Tetrapods which failed during a storm in 1980. The failure was mainly attributed to the breakage of large Tetrapods and was not related to the hydraulic instability. Also the interblock forces formed due to settlement and compaction of armors during the wave action caused the breakage of the units (Burcharth 1987).

The Akranes breakwater built with natural stones got severely damaged during a storm in 1980. Later two other storms during the period of 1980-1984 have hit the breakwater causing its catastrophic failure. The breakwater head and outermost section of about 55 m of the breakwater were washed out and down below high water level (Sigurdarson et al. 1995). The main reason for the failure of this breakwater was the combined wave actions during the storm. The waves were solitary type which first lifted the armor blocks from their position by buoyancy and momentum then washed them during downrush. Also, the shorter wind waves caused the damage by "pounding effects" which shook the blocks loose of bonds with other blocks thus weakening the structure (Ziedler et al. 1992).

The failure of breakwaters were not only limited to failure of armor units but also due to prevailing site conditions and other structural failures which added up in progressive damage of the breakwaters. Many other breakwaters failed due to similar conditions encountered along with some site specific conditions leading to the damage of breakwater. Breakwaters in Humbolt Jetty California (1976), Noyo Harbor Jetties (1980), El Djedid Port of Arzew (1999), failed due to geotechnical instability while others like Hirtshals Harbour (1973), Azzawiya Libya (1979), Jalali Oman (1983), failed due to loss of toe support and morphological changes (Magoon et al. 1974, Davidson and Markle 1976, Maddrell 2005).

1.4 REHABILITATION OF DAMAGED BREAKWATERS

The failure of structures gave birth to some safer structures like berm breakwaters, submerged breakwaters and reef breakwaters. Even the rehabilitation of the damaged breakwaters and protection of existing breakwater structures were accomplished with some of such safer structures. The rehabilitation work is mainly undertaken using berm breakwaters while protection is provided with any of the other two structures specified above. Some of the rehabilitation and protection works using berm breakwaters are explained in next few paragraphs.

As quoted by Juhl and Jensen (1995), around seventeen berm type breakwaters were built in Iceland, since 1983, of which ten were new structures and other seven were reinforcements or repairs of the existing breakwaters in the form of additional protection on seaside of caisson breakwaters or modifications of existing conventional breakwaters.

The failed breakwater of Akranes during 1980-1984 storms, explained earlier, was rehabilitated with a berm type breakwater with two layers of stones on top of the berm and one on the slope from berm to the crest (Sigurdarson et al. 1995).

A berm breakwater project on St. George, Alaska, USA was undertaken during 1980s. Before the completion, project was shut down due to storms in 1986-87 which was half completed. Survey of breakwater before and after the storm was undertaken revealed that the half completed breakwater had withstood the storm attack with only minor changes in profile (Juhl and Jensen 1995).

The Husavik harbor consisted of concrete caisson which was facing large scale overtopping of waves. A berm type rubble mound breakwater on seaside of the pier was constructed to protect it and minimize the overtopping (Sigurdarson et al. 1995). The pier in Blanduos has been protected with the construction of a berm breakwater.

The Tripoli harbor breakwater was re-designed during 1997 with Accropode as primary armor layer combined with an extended underwater berm. This was adopted to tackle venting and overtopping problems (Burcharth 1987).

The breakwater at Lajes, Azores located in mid-Atlantic Ocean had deteriorated due to attack of storms and has been repaired regularly from 1963 till 2003. In 2005 a project to develop a permanent solution to rehabilitate the breakwater was started. A composite berm structure with Core-Loc in the slope along with a wide reshaping berm at toe was compared with conventional Core-Loc armored structure. The composite berm structure was found to perform better than the conventional breakwater structure (Melby 2005).

An industrial estate built near sea in Gismeroy, a small island in city of Mandal, was nearly destroyed due to storms in May and October 2000. A berm breakwater protection structure with 10 Ton cover layer was designed after the storms to protect the estate from further damage (Lothe and Birkeland 2005).

A damaged breakwater at Codroy, Canada has been rehabilitated with a berm breakwater with 12 m wide berm (PIANC MarCom 1992). Berm breakwaters have been used extensively to rehabilitate the existing damaged breakwaters in Iran some of which are in Aboumusa, Suza, Lengeh and Rishehr (Kheyruri 2005).

Most of the berm breakwaters mentioned above after construction have been hit by severe storms and were found to be safe without experiencing much damage. It is clear from the above discussion that berm breakwaters have been successfully implemented in many projects around the world and are working efficiently. It can also be observed that even incomplete berm breakwaters are capable of withstanding storm which gives the impression that a damaged berm breakwater might also withstand storm up to certain extent without failing catastrophically.

1.5 NEED AND SCOPE OF THE PRESENT STUDY

As explained earlier, there is a need to develop a safe and economical breakwater, as it represents a significant portion of capital investment in a port. Further, the required size of stone cannot always be realized due to non-availability of stones or difficulty in transportation and one may have to think about artificial armor units which can be cast in-situ or pre-casted. From the previous section it is apparent that the rubble mound structures with both types of armor units have failed/damaged catastrophically due to various reasons. It was also observed that berm breakwater could be used as a new structure or for rehabilitating the damaged breakwater or protecting the existing structure. This demonstrates that the berm breakwater has got potential as a viable alternative to conventional breakwaters.

Further, Oumeraci (1984) as quoted by Hughes (1993) writes that in the case of rubble mound breakwaters, the wave structure interaction has not been completely explained and the stability of these structures cannot be accurately modeled mathematically. The design of rubble mound breakwater has to be therefore semi-empirical relying more on laboratory studies and field experience. This has made it difficult to arrive at a design, which is both, safe from structural standpoint as well as being favorable from economic point of view. This offers a window of opportunity to undertake further research in the area of design of berm breakwater.

In this regard, physical model study on the stability of stable reshaped berm breakwater constructed with concrete cubes as armor is taken up for the present research work. The study also involves the evolution of an optimum configuration of berm breakwater. The shape of armor units considered for present investigations is a cube. Cubes are selected because of its massive structure which satisfies the requirement of an armor unit particularly as an alternative to natural armor unit. The production of cubes is economical than other artificial armor units like Accropode and Core-Loc (Van der Meer 1999). Also, the damage in case of cube armors is not progressive as it is for other slender armor units with legs (Bakker et al. 2003).

1.5 ORGANIZATION OF THE THESIS

The information gathered from the work is presented in five chapters.

Chapter 1 introduces the development of breakwaters and artificial armor units. Further, it includes some examples of breakwater failure along with the scope of the present work.

Developments in the design, construction and performance of rubble mound breakwater and berm breakwater are surveyed in Chapter 2 which in turn leads to the selection of berm breakwater as a topic of the present research work.

Chapter 3 describes the formulation of the present research problem and objectives. It discusses model scale selection, physical models, and limitations of model testing and describes the methodology of the present experimental work.

Chapter 4 discusses the results obtained from the experiments on the stability of statically stable reshaped berm breakwater with concrete cube as armor unit.

Chapter 5 constitutes the conclusions drawn from the present study and future recommendations.

References follow up after uncertainty analysis and cost analysis which are presented in appendices I and II respectively. A list of publications based on the present research work and a brief resume are put at the end of report.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Breakwaters have been built throughout the centuries but their structural development as well as their design procedure is still under massive change. Breakwater design is increasingly influenced by environmental, social, aesthetical aspects and new type of structures are being proposed and built. New ideas and developments are in the process of being tested regarding breakwater layout for reducing wave loads and failures. The failure or significant damage of conventional breakwater, due to onslaught of extreme waves, may have disastrous consequences. The rehabilitation of damaged breakwater is extremely costly or in some cases impossible to rebuild the structure.

This chapter covers a comprehensive review of previous studies on various design concepts, impact of governing parameters, construction and other aspects influencing breakwater stability of conventional breakwater and berm breakwater.

2.2 RUBBLE MOUND BREAKWATERS

As explained in the previous chapter, rubble mound breakwater is a trapezoidal shaped heap of rock/concrete units. As per CEM (2006) “A mound randomly placed stones protected with a cover layer of selected or specially shaped concrete armor units built to protect shore areas of harbor anchorage or basin from the effects of wave action is called Rubble Mound Breakwater”.

A rubble-mound breakwater in its most simple shape is a mound of stones. However, a homogeneous structure of stones large enough to resist displacements due to wave forces like a reef structure, is very permeable and might cause too much penetration not only of waves, but also of sediments if present in the area. Moreover, large stones are expensive because most quarries yield mainly finer material (quarry run) and only relatively few large stones. As a consequence the conventional rubble-mound structures consist of a layered structure with core of finer material covered by big blocks forming the so-called secondary and primary armor layers. To prevent finer material of the core being washed out through the armor layer, a secondary armor layer called filter layer must be provided. Structures consisting of armor layer, secondary layer, and core are referred to as multilayer structures. Fig. 2.1 shows a multilayer rubble mound breakwater with a superstructure.

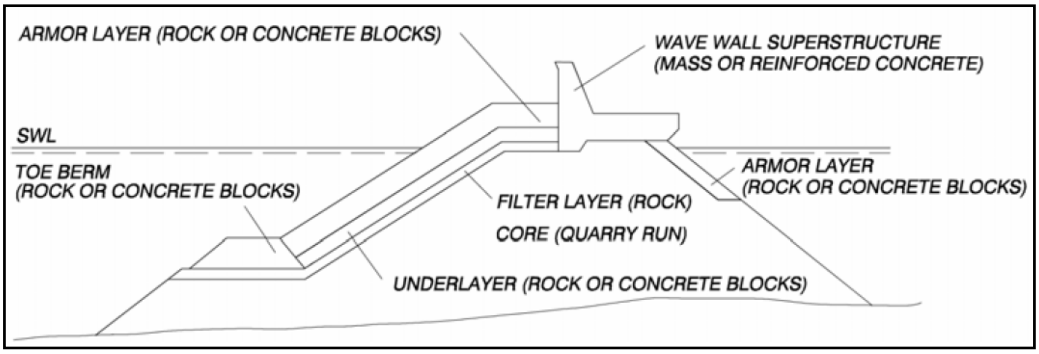


Fig. 2.1 Multilayer rubble mound breakwater with superstructure (CEM 2006)

The primary armor layer, directly exposed to the severe wave action, is made of strong and bulky stones. Apart from dissipating the wave energy, it also acts as a protective cover for secondary layer and core. The core usually consists of quarry run. The main use of the core material lies in forming a huge mass or heap to act as a barrier. The secondary layer acts as a cushion between primary layer and the core. The lower part of the armor layer is usually supported by a toe berm except in cases of shallow- water structures. Concrete armor units are used as armor blocks in areas with rough wave climates or at sites where a sufficient amount of large quarry stones is not available.

All the designers knew that for a breakwater to withstand high wave loads, larger stones were required. Until about 1930, the design of rubble mound was based only on general knowledge and experience of site conditions. There was no prevalent relationship between the heights of waves, slope of breakwater, size and density of armor stones. The breakwaters were built from the previous design and construction experience without caring for actual local conditions. The breakwater design was sometimes under designed or was over safe. Understanding of wave structure interaction is a primary requirement of any design of any marine structure. First attempt to relate the armor unit weight and other parameters was done by Spanish

Engineer Eqadro Castor in 1933. Although later several formulae were introduced, it was Hudson (1959) who through several model tests on breakwater sections subjected to regular waves evolved a simple design formula. Later, several modifications to his formula were made and other parameters were added. Even then, Hudson formula is still being used as a first approximation for calculating the armor unit weight because of its simplicity. The next section describes some of the formulae developed.

2.3 BREAKWATER DESIGN METHODS

In the design of rock armored structures, the median armor size required for stability constitutes the most important parameter to be determined. The design of rubble mound breakwaters essentially consists in determining the weight of the primary armor unit placed on a given slope, to be stable under design wave conditions. The other dimensions such as secondary armor and individual core material weight, crest width etc., are determined in terms of the primary armor unit weight and armor size.

Numerous researchers have found from their studies that the armor stability is a function of water depth, wave characteristics, armor unit weight, structure slope, porosity and storm duration. Each one of them investigated the structure stability with respect to different parameters (Hudson 1959, Ahrens 1970, Bruun and Gunbak 1976, Johnson et al. 1978, Ahrens 1984, Timco et al. 1984, Van der Meer and Pilarczyk 1984, Gadre et al. 1985, Van der Meer 1988, Hegde and Samaga 1996, Belfadhel et al. 1996). The formulae developed by various researchers are presented in Table 2.1. These formulae give the required weight of stone as a function of slope angle, wave height, and specific gravity of stones. CEM (2006, Part VI) provides the standard guidelines for the design and evaluation of rubble mound breakwaters.

The various formulae developed do not include all the parameters that influence the stability. Most of the earlier formulae were semi-empirical, developed on the basis of small-scale model tests conducted using regular waves. Later, with the ability to generate irregular waves in laboratory paved way for new formulae thereby representing the field conditions more accurately. Some of the important formulae developed are explained below.

Table 2.1 Various stability formulae (Poonawala 1993)

Author	Country	Formulae
CASTRO	SPAIN	$W = \frac{0.704}{(cot\alpha + 1)^2 \sqrt{cot\alpha - \frac{2}{\gamma_r}}} \frac{H^3 \gamma_r}{\Delta^3}$
IRIBARREN	SPAIN	$W = \frac{K}{(\mu cos\alpha - sin\alpha)^3} \frac{H^3 \gamma_r}{\Delta^3}$
EPSTEIN AND TYRREL	U.S.A	$W = \frac{K}{(\mu - tan\alpha)^3} \frac{H^3 \gamma_r}{\Delta^3}$
HUDSON	U.S.A	$W = \frac{1}{K_D cot\alpha} \frac{H^3 \gamma_r}{\Delta^3}$
HICKSON AND RODOLF	U.S.A	$W = \frac{0.0162}{tan^3\left(45^0 - \frac{\alpha}{2}\right)} \frac{H^2 T \gamma_r}{\Delta^3}$
LARRAS	FRANCE	$W = \frac{K\left[\frac{2\pi H/_L}{sinh(4\pi Z/_L)}\right]^3}{(cos\alpha - sin\alpha)^3} \frac{H^3 \gamma_r}{\Delta^3}$
BEAUDEVIN	FRANCE	$W = KK_s\left[\frac{1}{cot\alpha - 0.8} - 0.15\right] \frac{H^3 \gamma_r}{\Delta^3}$
HEDAR	SWEDEN	$W = \frac{K}{(cos\alpha - sin\alpha)^3} \frac{H^3 \gamma_r}{\Delta^3}$
SVEE	NORWAY	$W = \frac{K}{(cos^3\alpha)} \frac{H^3 \gamma_r}{\Delta^3}$
SN-92-60	USSR	$W = \frac{K}{\sqrt{1 + cot^3\alpha}} \frac{H^2 L \gamma_r}{\Delta^3}$
RYBTCHEVSKY	USSR	$W = \frac{K}{(cos^3\alpha)\sqrt{cot^3\alpha}} \frac{H^2 L \gamma_r}{\Delta^3}$
METELICYNA	USSR	$W = \frac{KK_s}{cos^3(23^0 + \alpha)} \frac{H^3 \gamma_r}{\Delta^3}$

2.3.1 Eqadro Castro formula

Eqadro Castro, a Spanish Engineer, first attempted to develop an equation correlating armor unit weight with the wave height. The equation is given as,

$$W = \frac{0.704}{(\cot\alpha + 1)^2 \sqrt{\cot\alpha - 2/\gamma_r}} \frac{H^3 \gamma_r}{\Delta^3} \quad \text{....................................... (2.1)}$$

Where,

W is weight of individual armor unit in metric tons, H is design wave height in meters, γ_r is specific weight of armor unit in ton per cubic meter, α is angle of breakwater slope and Δ is relative mass density of armor.

This formula was based on theoretical assumption that destructive action of wave is proportional to its energy and the stability of units under wave action is inversely proportional to a function of the angle of slope. This formula yielded very small values of W and engineers rejected this formula thus it was not used for practical application.

2.3.2 Hudson formula

Hudson (1959) developed a simple expression for the minimum armor weight required for a given wave height. The stability formula was developed by equating the total drag and inertial force of wave to the relevant component of the weight of the individual unit. This may be written in terms of the armor unit mass (W) and design wave height (H).

$$W = \frac{1}{K_D \cot\alpha} \frac{H^3 \gamma_r}{\Delta^3} \quad \text{.. (2.2)}$$

Where,

W is weight of individual armor unit in metric tons, K_D is stability coefficient which varies with the type of armor unit and H is design wave height in meters.

A range of wave heights and periods were studied. In each case, the value of K_D corresponds to the wave condition representing worst stability condition. The K_D value mainly depends on the shape of armor unit which includes friction between the units.

Some of Hudson's conclusions were:

i. The formula could be used more accurately for no damage and no overtopping criteria. No damage means that a maximum of one percent of armor stones can only be displaced from their initial position when subjected to wave attack of design wave height.

ii. K_D equal to 3.2 (non-breaking condition) is adequate for angular quarry stone armor, which includes the safety factor also.

iii. The stability of rubble mound breakwater is not significantly affected by the variation in relative depth (d/L) and wave steepness (H/L) ratios.

With the improvement in the investigation techniques, better insight into the wave-structure interaction and the development of measurement instruments, many investigators have pointed out that certain aspects of Hudson's formula needed improvement or updating. These can be briefly put under:

a. Hudson's formula was developed considering regular waves and does not include the effect of irregular waves. He also has not specified the wave height of irregular wave train to be considered for the design.

b. The formula was developed to represent the behavior of natural rock type armor units which basically depend upon their weight for stability under wave attack. The validity of the formula for artificial armor units depending primarily on interlocking for stability (e.g. Dolos) is thus a debatable point.

c. The influence of the variables such as wave period, duration of the storm, degree of overtopping, damage history, friction, porosity of armor units, permeability of core and randomness of incident waves were not considered in his stability equation. Therefore it was argued that the value of K_D is too simple to represent the stability of armor units and should only be accepted as a first approximation Bruun and Gunbak (1976). Further wave steepness, which has a significant role in determining the stability, was also neglected by Hudson (1959).

In spite of the above drawbacks, Hudson's formula is the simplest and widely adopted means of obtaining a good preliminary estimate of the weight of armor units. It requires only two quantities which need to be estimated, i.e. wave height and K_D. However, the design has to be substantiated by the model testing before finalizing the cross section of a breakwater.

2.3.3 Van der Meer equations

Van der Meer (1988) derived formulae which include the effect of random waves, wave periods and a wide range of core / underlayer permeability. The investigations by Van der Meer showed clear dependence of wave period on erosion damage. Two design formulae were obtained in terms of Hudson's stability number considering the tests performed by him and the results of Thompson and Shuttler (1975).

For plunging waves,

$$\frac{H_S}{\Delta D_{n50}} = 6.2 P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^{-0.5} \qquad (2.3)$$

For surging waves,

$$\frac{H_s}{\Delta D_{n50}} = 1.0\, P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} (\cot \alpha)^{0.5} \xi_m^{P} \qquad (2.4)$$

Where, the parameters not previously defined are:

H_s = Significant wave height

Dn_{50} = Nominal diameter of the armor unit

P = Permeability of the structure

S = Design damage number = A_e/D^2n_{50}

A_e = Erosion area from profile

N = Number of waves

ξ_m = Iribarren number = $\tan\alpha / S_m^{1/2}$

S_m = Wave steepness for mean period = $2\pi H_s/gT_m^2$.

T_m = Mean wave period

And the transition from plunging to surging waves is calculated using a critical value:

$$\xi_m = \left(6.2P^{0.31}\sqrt{\tan\alpha}\right)^{1/(P+0.5)} \quad \text{……………………………..…..…… (2.5)}$$

These formulae developed by Van der Meer included the parameters such as wave period, storm duration, random wave conditions and a clearly defined damage level. The starting level of damage, S = 2 to 3, is equal to the definition of no damage in the Hudson's formula. Van der Meer considered storm duration of 5000 to 7000 waves during his investigations. Van der Meer formulae, though versatile, parameters like permeability are difficult to evaluate and quantify to a degree of accuracy required for design. This has limited the use of this new formula for the design of breakwaters.

2.3.4 Other design formulas

2.3.4.1. Sherbrooke University Formula

Sherbrooke University Formula (Belfadhel et al. 1996) is derived from Hudson's formula by replacing K_D with actual percentage of damage within the active zone. This was developed for regular waves on both steep (1.5:1) and flat (2.5:1) slopes.

$$W_{50} = \frac{1.88\gamma_r H_D^3}{\Delta^3 \cot\alpha^{2.31} Pd^{0.6}} \quad \text{.. (2.6)}$$

Where, H_D is design wave height and Pd is percent damage within the active zone delimited by an elevation of one wave height below and above the water level.

2.3.4.2 Koev Formula

Koev (1992) derived a stability equation based on statistical analysis of 21 formulas developed earlier for breakwater armor design using regular waves. This formula is also similar to Hudson's formula with K_D replaced by wave steepness (H/L).

$$W_{50} = \frac{0.1421\gamma_r H_D^3}{\Delta^3 \cot\alpha^{1.5667}\left(H/L\right)^{0.3843}} \quad \text{....................................... (2.7)}$$

The equation is valid for wave steepness ranging from 0.04 and 0.1 and slope ($\cot\alpha$) between 1.1 and 20.

2.3.4.3 Kajima's Stability Formula

Kajima (1994) proposed the following expression of stability number applicable to Tetrapods for horizontally composite breakwaters (Hanzawa et al. 1996).

$$N_s = H_{1/3}/\Delta D_n = 8.5\left(S / N^{0.5}\right)^{0.16} \xi^{-0.5} \quad \text{...} \ (2.8)$$

Hanzawa et al. 1996 tested the above equation with their model test results and found no good agreement. The reasons for the disagreement given by them were, the equation was formulated to cover a range of relatively high damage level which is usually not seen in the ordinary design of port structures, and the water depth of the structure Kajima investigated was greater than their test data which resulted in a bigger cross section affecting the parameter S.

2.3.4.4 Melby and Hughes formula

Melby and Hughes (2004) used the maximum depth integrated wave momentum flux (N_m) to describe the armor stability. The new equations explicitly included the effects of nearshore wave height, wave period, water depth, and storm duration as well as the characteristics of wave breaking on the structure (Melby, 2005).

$$N_m = 5.0\,P^{0.18}\left(S/\sqrt{N_z}\right)^{0.2}\sqrt{\cot\theta} \qquad s_m \geq s_{mc} \ \text{..........................} \ (2.9)$$

$$N_m = 5.0\,P^{0.18}\left(S/\sqrt{N_z}\right)^{0.2}\left(\cot\theta\right)^{0.5-P} s_m^{-P/3} \qquad s_m < s_{mc} \ \text{...........................} \ (2.10)$$

where $s_{mc} = -\,0.0035 \cot\theta + 0.028$

$$N_m = \left\{\frac{K_a\left[\left(M_F\right)_{\max}/\gamma_w h^2\right]}{(S_r - 1)}\right\}^{1/2} \frac{h}{D_{n50}} \ \text{...} \ (2.11)$$

With K_a = 1. Here M_F = wave momentum flux, s_{mc} = critical wave steepness on the structure, P = structure permeability, S = damage level, γ_w = water specific weight, N_z = t/T_m = number of waves at mean period during event of duration t. T_m = mean wave period, $s_m = H_s/L_m$ = wave steepness, H_s = significant wave height, L_m = wavelength based on mean wave period, and θ = seaward structure slope from horizontal.

$$\text{Also, } \left[\frac{M_F}{\rho g h^2}\right]_{\max} = A_o\left[\frac{h}{gT^2}\right]^{-A_1} \ \text{...} \ (2.12)$$

Where, $A_o = 0.6392\left[\frac{H}{h}\right]^{2.0256}$ and $A_1 = 0.1804\left[\frac{H}{h}\right]^{-0.391}$

2.3.4.5 Van Gent formulae (2003)

Van Gent et al. (2003) gave a single simple stability formula than the Van der Meer formulas (1988). The influence of wave period was found to be small and hence they did not consider it in their equation. The permeability parameter is replaced by a structure parameter i.e., diameter of core material. The equation is given as:

$$\frac{H_s}{\Delta D_{n50}} = 1.75\sqrt{\cot\alpha}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\left(1+\frac{D_{n50core}}{D_{n50}}\right)^{2/3} \quad \text{.......................................} \quad (2.13)$$

The influence of filters in considering the permeability was neglected. The equation is limited to following conditions:

$H_s/\Delta D_{n50}$ = 0.6 – 4.3; ξ_m = 1.0 – 5.0; $D_{n50core}$ /D_{n50} = 0 – 0.3; and slope = 1:2 to 1:4.

2.3.5 Improvements in breakwater design

Along with physical model studies few researchers (Haan 1989, Hegde 1996, Mase et al. 1995, Melby 2005, Kim et al. 2007, Mandal et al. 2007, Balas et al. 2010, Van Gent et al. 2011) had started developing mathematical and numerical models for the design of breakwater structures.

Kobayashi and Jacobs (1985) developed a mathematical model to predict the flow characteristics in the downrush of regular waves and the critical condition for initiation of movement of armor units. The effect of bottom friction, permeability, water depth and wave overtopping were not considered for modeling. They concluded that the mathematical model could be a supplement for hydraulic model tests and empirical formulas.

Haan (1989) developed a computer package for optimum design of rubble mound breakwater. The program included all the parameters required for design and the armor weight was calculated based on Van der Meer formula.

Mase et al. (1995) examined the applicability of neural network for prediction of stability of rubble mound breakwaters. They found good agreement between the predicted and measured stability numbers.

Hegde (1996) also developed a mathematical model called DORUB to predict the armor stone weight of a rubble mound breakwater using the quarry yield optimally. He used programming language C and Visual basic to draw the cross section.

Yagci et al. (2005) developed three ANN models and a fuzzy logic model to study the breakwater damage ratio. Damage ratio is defined as the ratio of the displaced armor to total armor units in the section. It was observed that all the four models predicted the damage ratio fairly well compared to the experimental values. It was suggested that ANN performance can be further enhanced with the extension of training data set by considering the data for all slopes.

Kim et al. (2007) used Probabilistic Neural Network (PNN) to predict the stability number of breakwater and assess its performance. It was proved that PNN provided more advanced results than the empirical model and ANN in estimating the stability number of breakwaters.

Mandal et al. (2007) also have concluded that the neural network can predict the stability of a breakwater better than the empirical formula with higher correlation coefficient for ANN models compared to Van der Meer formula. Some upgraded algorithms like LMA (Levenberg-Marquardt Algorithm), SCG (Scaled Conjugate Gradient) and GDA (Gradient Descent with Adaptive learning rate) were also tested for enhancing the prediction capability of stability and found that LMA showed better improvement compared to other two algorithms. The network developed predicted a relatively lesser armor weights compared to empirical model.

Etemad-Shahidi and Bonakdar (2009) developed a M50 machine learning method for designing a rubble mound breakwater. It was observed that the performances of the proposed model trees were better than those of previous empirical and the soft computing methods in predicting the stability of rubble mound breakwater.

Artificial neural networks based on principal component analysis, fuzzy systems and fuzzy neural networks can be adopted for preliminary design of rubble mound breakwaters as suggested by Balas et al. (2010). However, they have emphasized the hydraulic model tests for the final design, since; the safety of coastal structures is highly variable and depends upon the unpredictable nature of wave-structure interaction.

2.4 STABILITY OF RUBBLE MOUND BREAKWATER

The main governing parameter of the armor layer stability is the stability number. The stability of a breakwater is derived from Hudson's formula which can be rearranged to get,

$$N_s = \frac{H_s}{\Delta D_{n50}} = \left(K_D \cot \alpha\right)^{1/3} \text{ .... (2.14)}$$

Where, N_s = Hudson's stability number,

$$D_{n50} = \left(\frac{W}{\gamma_r}\right)^{1/3} \text{ .... (2.15)}$$

The wave height commonly used in this formula is the significant wave height, $H_{1/3}$ or H_{mo} (H_{mo} is the estimate of significant wave height deduced from spectral information). The value of N_s is usually established by testing the breakwater until its damage reaches equilibrium. To attain this equilibrium, the time series of the sea state may have to be recycled many times. The number of times it is recycled is also an important parameter to be taken into account.

Van der Meer (1988) gives the examples of types of structures with corresponding H/ΔD values as:

- H/ΔD < 1, Caissons or seawalls
- H/ΔD = 1 – 4, Stable breakwaters
- H/ΔD = 3 – 6, S- shaped and berm breakwaters
- H/ΔD = 6 – 20, Rock slopes / beaches
- H/ΔD = 15 – 100, Gravel beaches
- H/ΔD > 500, Sand beaches and Dunes

2.4.1 Factors affecting the stability of non-overtopping breakwater

The factors affecting the stability can be classified under environmental conditions and structural conditions as coined by various authors (Hudson 1959, Van der Meer 1988, Losada 1990, Bruun 1981, CEM 2006). After reviewing the literature on the breakwaters, it is observed that large number of parameters affect the stability and hence the design of breakwaters. The various factors affecting the stability of breakwaters are given below.

a) Environmental Variables
   i. Wave characteristics
   ii. Water depth
   iii. Duration of wave attack
   iv. Wave run-up and run-down
b) Structural Variables
   i. Geometry of the breakwater
   ii. Permeability of the structure
   iii. Method of construction
   iv. Foundation

2.4.1.1 Wave characteristics

Wave characteristics of a wave include wave height and wave period. The important parameter which affects stability is the wave height. The wave height used in most of the design formulas is the wave height measured in front of the structure and the armor stone weight required in conventional breakwater is directly proportional to H^3. The wave forces of higher waves caused instability of armor units thus indicating that generally damage increases with increasing wave height.

Wave period is another parameter which affects the stability indirectly through some related parameters like wave steepness, wave breaking characteristics, etc. Wave period was not considered by Hudson in his formula. The wave period is often described by a dimensionless variable, wave steepness (s) or surf similarity parameter (ξ). Wave steepness, s, can be defined by the ratio of deep water wave height to the deep water wave length and is given as,

$$s = \frac{2\pi H}{gT^2} \qquad (2.16)$$

Iribarren defined the similarity parameter, ξ, relating wave steepness, s to the slope angle of the structure, $\tan\alpha$, given as,

$$\xi = \frac{\tan\alpha}{\sqrt{s}} \quad \text{..} \quad (2.17)$$

Ahrens (1970) conducted studies to study the influence of breaker type on stability of breakwater and found that for collapsing breaker the stability is lowest.

Ergin and Pora (1971) justify the use of H_s of an irregular wave train as the height of a regular wave train in the laboratory**.**

Battjes (1974) found that type of breaker has a major influence on stability of breakwater and he classified the breaker type based on offshore surf parameter as:

Breaker type	**Limiting criteria**
Surging or collapsing	$3.3 < \xi_o$
Plunging	$0.5 < \xi_o < 3.3$
Spilling	$\xi_o < 0.5$

For a fixed slope, breakers will change form from collapsing towards spilling as steepness increases (Bruun and Gunbak 1976).

Van der Meer and Pilarczyk (1984) have concluded that the influence of the wave period is evidently larger for breaking waves (ξ < 2.5 - 3.5) than for non-breaking waves (ξ > 2.5 - 3.5).

Thompson and Vincent (1985) developed an empirical method to relate statistical and energy based significant height estimates. They concluded that monochromatic waves are distinctly different physical phenomenon than regular waves. There is no intrinsic relationship between monochromatic wave height, H and the irregular parameter, H_s and energy based wave height parameter, H_{mo}. H_s and H_{mo} are parameters of different attributes of the same physical phenomenon and H, (regular wave height) is a parameter of an entirely different physical phenomenon. If for simplicity linear wave conditions are assumed, $H = H_s$, the monochromatic train will contain twice as much energy as the irregular wave train. Conversely, if energy is most important, H could be chosen so that $H = 0.71H_s$, which means many waves in the irregular train will be higher than the monochromatic height (Thompson and Vincent 1985).

The study of Ahrens (1975) as reported by Van der Meer (1988) in large wave tank showed the importance of the wave period on the stability of riprap. The tests were performed with the regular waves.

Pilarczyk and Zeidler (1996) gave the critical values of inshore surf similarity parameter, ξ_b for different types of breaker for plane profile as:

Breaker type	**Limiting criteria**
Surging	$3.0 – 3.5 < \xi_b$
Collapsing	$2.0 < \xi_b < 3.0 – 3.5$
Plunging	$0.5 < \xi_b > 2.0$
Spilling	$\xi_b < 0.5$

Vidal et al. (2006) based on their experimental study have shown that wave height parameter H_{50}, defined as the average wave height of the 50 highest wave reaching a rubble mound breakwater in its useful life, can describe the effect of the wave height on the history of the armor damage caused by the wave climate during the useful life of the structure.

2.4.1.2 Water depth

Water depth in front of structure has a major effect on its stability. If depth of water is small then waves shoal and loose their energy due to bottom friction but deeper water can sustain higher waves which can break close to the breakwater structure resulting in increased wave run-up and run-down causing higher damage to the structure.

Weight of armor may be reduced by one-fourth at depth below one-third of the toe depth and by three-fourth below two-third depth (Palmer and Walker 1976).

As reported by Van der Meer (1998) from Van der Meer (1988) for depth limited condition it is suggested to use $H_{2\%}$ rather than H_s to describe the stability of breakwater.

2.4.1.3 Duration of wave attack

Font (1968) found that the damage generally increased as the duration of exposure increased. But, it was also observed that for shorter duration of wave exposure the amount of damage was high indicating that the shorter exposure might improve the interlocking thus making the structure more resistant to longer exposure of the same wave.

Van der Meer and Pilarczyk (1984) have concluded that with random waves a stable profile could not be found with less than 10,000 waves where as for regular waves equilibrium profile is generally found within 1000 - 2000 waves.

Hall (1994) in his experiments on dynamically stable breakwaters considered a storm duration of 3000 waves. This was based on the studies previously done by Hall and Kao (1991).

Hegde (1996) concluded that the conventional breakwater will have a stable profile after about 3000 waves.

2.4.1.4 Wave run-up and run-down

The wave action and breaking on the slope cause wave run-up and run-down (Fig. 2.2). Run-up and run-down heights are defined as the vertical distance from the still water level to the crest and trough of the up- and down-rushing wave.

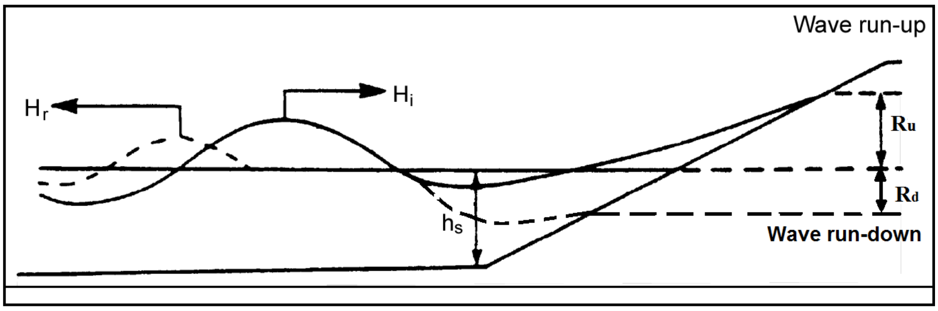


Fig. 2.2 Definition sketch of Wave Run-up (R_u) and Wave run-down (R_d)

The effects of the permeability and the roughness of the rubble-mound face are important factors in wave run-up (Hudson 1958). For a given structure slope as the wave steepness decreases, the relative wave run-up increases (CERC 1966).

Saville (1957) suggested an iterative method for calculation of wave run-up on composite slopes due to regular waves. The method proposed to consider an equivalent straight slope from the bottom to the point of maximum initial run-up on the structure. The initial run-up is estimated for the initial bottom slope of the breakwater. This process is repeated till same values of run-up are obtained between two successive iterations. Saville limited the application of composite- slope method for berms with a length less than L/4, where L is the design wavelength for the structure.

As quoted by Bruun and Gunbak (1976), Saville (1953) has concluded that effect of water depth is negligible when d/H > 3.0 for all wave steepness. Wave run-up increases with increasing ξ values and trends gets milder at high ξ values. Run-down also increases with increasing ξ values and becomes nearly constant at high ξ values ($\xi > 4.0$). Higher run-down was observed for $\cot\alpha = 3.0$ than for $\cot\alpha = 2.0$ at the breaking range when $\xi < 3.0$ (Bruun and Gunbak 1976). The reason quoted by them is that the water running up and down on the 1 in 3 slope travels a longer distance than on the 1 in 2 slope which may cause a higher possibility of penetration of water deep into the breakwater body and thus cause a deeper run-down on the 1 in 3 slope.

For breakwaters of slope 1:1.5, maximum run-up could be up to 1.8 times depth of water (Palmer and Walker 1976).

Losada and Gimenez-Curto (1981) gave a general form of wave run-up and run-down studying the data corresponding to Ahrens and McCartney (1975) and Gunbak (1979). They found that wave run-up, run-down and surf similarity parameter is exponentially related as,

$$\frac{R_u}{H} = A\{1 - \exp(B.\xi)\} \quad \text{.......} \quad (2.18)$$

$$\frac{R_d}{H} = A\{1 - \exp(B.\xi)\} \quad \text{.......} \quad (2.19)$$

The coefficients A and B are to be fitted by Least-square method.

Various graphs were provided by Shore Protection Manual (SPM 1984) for calculation of wave run-up on a slope due to regular waves. For irregular waves a method suggested by Ahrens (1977) was adopted.

Armor damage due to displacement occur mainly due to run-down for steep slopes and run-up is damaging for flatter slopes i.e., $\cot\alpha > 3.5$ (Sorensen and Jensen 1986).

Leenknecht et al. (1992) mentioned an empirical formula given by Ahrens and McCartney (1975). The related the run-up and surf similarity parameter by a non-linear relationship.

$$\frac{R_u}{H_i} = \frac{a\xi}{1+b\xi} \quad \text{(2.20)}$$

Where, a, b are empirical coefficients associated with the type of armor unit. The values are listed in Table 2.2.

Table 2.2 Values of wave run-up coefficients for rough slope (Leenknecht et al. 1992)

Armor material	**a**	**b**
Riprap	0.956	0.398
Rubble (Permeable – No core)	0.692	0.504
Rubble (2 layers – Impermeable Core)	0.775	0.361
Modified Cubes	0.950	0.690
Tetrapods	1.010	0.910
Dolosse	0.988	0.703

Van der Meer and Stam (1992) studied the effects of various parameters viz. wave height, wave period, slope angle, water depth, structure permeability and spectral shape on wave run-up on smooth and rock slopes. An empirical relationship between wave run-up and surf similarity parameter was developed given as,

$$\frac{R_{ux}}{H_s} = a\xi_m \quad \text{for } \xi_m \leq 1.5 \quad \text{(2.21)}$$

$$\frac{R_{ux}}{H_s} = b\xi_m^c \quad \text{for } \xi_m \geq 1.5 \quad \text{(2.22)}$$

The coefficients of wave run-up levels are given in Table 2.3.

Table 2.3 Coefficients of wave run-up levels

Level (%)	**a**	**b**	**C**
0.13	1.12	1.34	0.55
1	1.01	1.24	0.48
2	0.96	1.17	0.46
5	0.86	1.05	0.44
10	0.77	0.94	0.42
Significant	0.72	0.88	0.41
Mean	0.47	0.60	0.34

Hughes (2004) considered momentum flux parameter and developed an empirical formula to compute wave run-up for both regular and irregular waves. Hughes stated that existing wave run-up formulas for regular waves based on Iribarren number performed well for milder slopes but poorly for steeper slopes.

$$\frac{R}{h} = CF(\alpha)\left[\frac{M_F}{\rho g h^2}\right]^{1/2} \quad \text{...} \quad (2.23)$$

Where, C is an unknown constant, F(α) is a function of slope angle to be determined empirically and M_F = wave momentum flux.

The Hughes (2004) formula was applied to old regular wave data of Granthem (1953) and Saville (1955) and following relation was obtained.

$$\frac{R}{h} = 3.84\tan\alpha\left[\frac{M_F}{\rho g h^2}\right]^{1/2} \quad \text{...} \quad (2.24)$$

Influence of scale effect on wave run-up was studied by Schüttrumpf and Oumeraci (2005) and found that their effect is negligible under usual model conditions.

Muttary et al. (2006) found that wave run-up is closely related to clapotis height in front of the breakwater. An empirical equation relating wave run-up and reflection coefficient was developed given by,

$$R = aH_i(1 + C_r) \quad \text{...} \quad (2.25)$$

Where, ‘a’ is a coefficient and its value is 1.31 for regular and 1.17 for irregular waves. H_i is the incident wave height, H_r is the reflected wave height and C_r is reflection coefficient ($C_r = H_r/H_i$). The Eq. 2.24 is applicable for non-breaking waves and little or no-overtopping conditions. The empirical coefficient ‘a’ between 1.2-1.3 is only applicable for 1:1.5 slopes.

Wave run-down on rock slopes, as per CEM (2006), is given by

$$\frac{R_{d2\%}}{H_s} = 2.1\sqrt{\tan\alpha} - 1.2P^{0.15} + 1.5e^{-(60s_{om})} \quad \text{...............................} \quad (2.26)$$

2.4.1.5 Geometry of breakwater

The thickness of the structure is important for wave energy absorption as reported by Hunt (1959). It was found that wave energy absorption by the structure continued up to a thickness of 2d, where d is the water depth.

Losada & Gimenez-Curto (1981) concluded that for each type of armor unit, an optimum slope of maximum stability exists. The greater the interlocking among armor units, the steeper the optimum stability slope.

Van der Meer (1987, 1988) has concluded that if fine material can’t erode through the armor layer then the stability of the armor layer is not influenced by the grading and size of filter layer.

Hall (1987) as quoted by Hall and Kao (1991) concluded that thicker armor layer is very effective in reducing internal differential pressure in a stable breakwater.

The trapezoidal shape was found to be optimum for rubble mound breakwaters (Pilarczyk and Zeidler 1996).

2.4.1.6 Permeability of the structure

The porosity of the armor layers, underlayers, core and filter materials is an important factor in the stability of a rubble structures. The permeability/porosity of the structure has large influence on stability as shown by Hudson (1958), Hedar (1986), Thompson & Shuttler (1976), Bruun and Gunbak (1977), Van der Meer (1988), Hegde (1996).

Hudson (1958) found that permeability and the roughness of the rubble-mound face are important factors in wave run-up.

Van Gent (1992) in his report derived some equations to describe the motion of wave on permeable structures and developed a model based on these equations. The model developed was tested on permeable and impermeable berm breakwater and submerged breakwater.

Hegde and Srinivas (1995) experimentally found that core porosity affects stability and wave run-up of a structure. With the increased porosity the stability was found to increase and wave run-up was found decreasing. The reasons for such a behavior as quoted by Hegde and Srinivas (1995) were large inflow and high energy dissipation within the core of the structure.

2.4.1.7 Method of construction

Randomly placed armor stones or pell-mell construction would have less stable compared to those placed with special care. The method of placing units randomly, affects the stability and repeatability of tests was proved by Thompson and Shuttler (1976) during their experiments.

Very often rubble mound breakwater design seems to be a result only of stability considerations corresponding to design wave conditions. Designers tend to put too little emphasis on practical problems related to construction, maintenance and repair. Based on the studies by Burcharth and Rietveld in 1987 on the different methods of construction process have found that during construction of seaward slope of breakwater, each armor block takes about the same time to place, independent of its weight over a fairly wide range, and savings can be achieved by placing fewer and heavier concrete blocks on a steeper slope than a relatively lighter natural stones over a gentler slope depending upon the site conditions and wave climate.

Palmer and Walker (1976) investigated type of placement, on the stability. The required weight of loosely placed stone may be two times that of a well placed stone, and keyed and fitted armor is several times more stable than loosely placed armor.

2.4.1.8 Foundation

Breakwater over a weak foundation is less stable and has an undue settlement. If the underlying material is too weak to withstand the superstructure, wedges of material will be displaced which might result in uneven settlement due to which there will be instability to the rubble mound. PIANC (1976) suggested some methods to strengthen the foundation and improve the stability of rubble mounds like, removal of unsuitable material or replacing it by sand or gravel, increasing the stability by providing sand columns etc. Many researchers also suggest alternative structures like berm breakwater, pile breakwater, submerged structure etc.

2.5 DAMAGE OF RUBBLE MOUND BREAKWATERS

'Damage' quantification is essential for the qualitative and quantitative analysis of stability of rubble mound structures. There are various types of damage to breakwater that will lead to its failure. Fig. 2.3 shows the different failure modes of a rubble mound breakwater. The armor unit instability is the most critical among all the failures since its causes the disintegration of armor layer and further progressive failure making the breakwater unstable (Kamali and Hashim 2009). This section is mainly concentrated on damage of the breakwater due to armor layer instability.

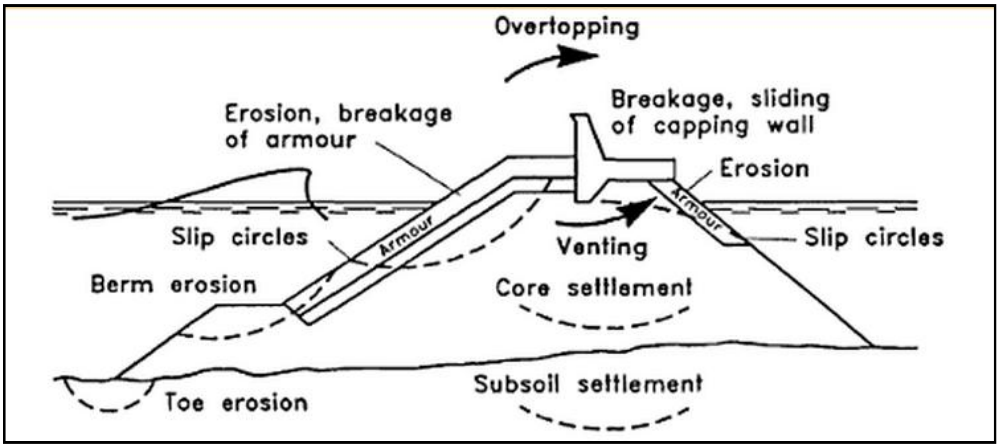


Fig. 2.3 Failure modes of a rubble mound breakwater (Burcharth and Liu 1995)

As quoted by Bruun (1985), Iribarren defined that a rubble mound reached its failure when the armor units of the secondary layer are exposed. Hudson (1959) defined no damage condition as maximum 1% of displacement of armor stones from its initial position. The percentage of damage is the number of displaced stones divided by the number of stones in the attacked area (SWL + H and SWL – H) times hundred as per Van de Kreeke (1969).

If the number of units seems to rock and move a distance which was less than the overall size of the unit, it was considered as stable damage, if the distance was greater than that size, it was considered as unstable damage according to Ouellet (1972).

Thompson and Shuttler (1976) defined a damage parameter N_{Δ} as 'the number of D_{n50} sized spherical stones eroded from the slope which was obtained by dividing the product of the bulk density, and the eroded volume by the size of a spherical stone'.

$$N_{\Delta} = \frac{D_{n50} \times Ae \times 9\rho_b}{\rho_a \frac{\pi}{6} D_{n50}^{\ 3}} \quad \text{...} \quad (2.27)$$

where, ρ_a = mass density of armor units, ρ_b = bulk density of armor units, D_{n50} = nominal diameter of the armor units. The damage parameter is independent of the size of the armor layer (i.e., length above and below water level and thickness) and also N_{Δ} is directly related to the erosion area and stone size. The measurement of the bulk density in prototype is one of the major disadvantages of this formula.

'Damage' is also represented as damage parameter, D%, defined as percentage of armor units displaced from the cover layer by the wave action (Hudson 1958). CEM (2006) revised this definition with respect to rock and concrete armor units. For rock it is taken as percentage of eroded volume and for concrete units as percentage of units more than D_n within some restricted area around Still Water Level (SWL). This specific zone around SWL is also called active zone. In case of emerged breakwaters, most displacements occur approximately in the area from one H_s below SWL to one H_s above SWL and the number of units placed in this zone is often used as the reference number (CEM 2006). The area between the middle of the crest to one H_s below SWL was considered as active zone in the Shore Protection Manual (SPM 1984). These definitions of active zone are not applicable to Low Crested Structures or submerged barriers. Moreover, due to different designs, total number of armor units differs for each structure; and varying definition of damage computed through various studies cannot be compared. The range of values of damage parameter for different armor units are given in CEM (2006) are presented in Table 2.4.

Table 2.4 Damage parameter "D" for two-layer armor (CEM 2006)

Unit	**Slope**	**Initial Damage**	**Intermediate Damage**	**Failure**
Rock	1:2 – 1:3	0 – 5%	5 – 10%	> 20%
Cube	1:1.5 – 1:2	--	4%	--
Dolosse	1:1.5	0 – 2%	--	≥ 15%
Accropode	1:1.33	0%	1 – 5%	≥ 10%

Van der Meer (1988) used a relation to measure the damage in terms of area of erosion (Refer Fig. 2.4), which is as follows,

Damage level, $S = \frac{A}{D_{n50}{}^2}$ .... (2.28)

Where, A_e is the erosion area and D_{n50} is the nominal diameter of the stones.

The limit of damage (S), for rocks, observed by Van der Meer (1988) is given in Table 2.5.

Table 2.5 Van der Meer damage criteria

Slope	**Initial damage**	**Intermediate damage**	**Failure**
1:1.5	2	3 – 5	8
1:2	2	4 – 6	8
1:3	2	6 – 9	12
1:4 to 1:6	3	8 – 12	17

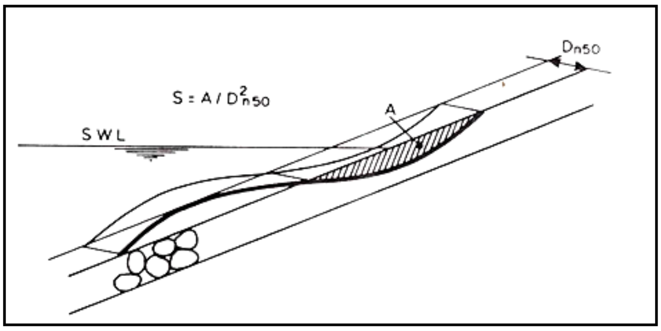


Fig. 2.4 Erosion area and damage level, S (Van der Meer 1988)

Relative damage number, N_{od}, is defined as the number of units displaced within a vertical strip of width D_n (D_n = equivalent armor size) stretching from the bottom to the top of the armor layer (Van der Meer 1988). This relative damage number is generally used to define damage of artificial armor units. This definition of damage can be easily related to a percentage of damage. N_{od} gives the actual damage, which is related to the number of units in a cross section with a width of D_{n50}. Table 2.6 gives the limit of the N_{od} values for initial damage condition and failure of the structure with different armor units. The disadvantage of N_{od} is its dependency on the slope length (CEM 2006).

$N_{od} = \frac{No.of\ units\ displaced\ \ out\ of\ armour\ \ layer}{Width\ of\ test\ section\ /D_{n50}}$.. (2.29)

Table 2.6 Damage level by N_{od} for two – layer armor (CEM 2006)

Units	Slope	Initial damage	Failure
Cube	1:1.5	0	2
Tetrapods	1:1.5	0	1.5
Accropode	1:1.33	0	0.5

Van der Meer (1988) experimentally determined relationship between N_{od} and S for different armor units as,

Cubes, Slope 1:1.5, N_{od} = (S-0.4)/1.8

Tetrapod, Slope 1:1.5, N_{od} = (S-1)/2

Accropode, Slope 1:1.33, N_{od} = (S-1)/2

Since profile measurement is not commonly adopted for damage calculation of artificial armor units, damage level (S) cannot be used for them to quantify the damage (Bakker et al. 2009).

Unit movement may take place in different ways and each has different contribution to damage. Some of the units may displace out of layer and completely fail to perform their function. While, some units displaced from their original position may still remain in the eroded area and reach a stable position. In this case, the displaced units may still contribute effectively to the slope protection by still remaining in the slope covering the secondary layer. Counting method considers all the displaced units as damage regardless of their new position, while profile measurement does not take the units displaced but remains in the eroded area as damage. To reach a conclusion, it should be considered that any type of unit displacement reduces the layer integrity. Hence, counting method can lead to damage overestimation on one hand, and profiling method may gives underestimated damage on the other hand.

Gómez-Martín and Medina (2013) adopted a new method of damage calculation which was suggested by Gómez-Martín and Medina (2006). The new method was named as 'Virtual Net Method'. In the Virtual Net method a virtual net will be projected over a photographed armor layer (Fig. 2.5). The armor layer will be divided into strips of a constant width. The width ($a=m*D_n$) and length ($b=k*D_n$) of each strip are calculated. The number of armor units within each strip (N_i) is counted, and the porosity of each strip is estimated using Eq. 2.30, before and after the wave attack. Next, the dimensionless armor damage in each strip (D_i) is calculated using Eq. 2.31. The overall damage is then calculated using Eq. 2.32. The virtual net method considers armor unit extraction, armor layer sliding as a whole. The applicability of the virtual net method in case of berm breakwaters is not viable as projecting a virtual net over the photographed armor layer is difficult because of presence of berm.

$$p_i = 1 - \frac{N_i D_{n50}^2}{ab} = 1 - \phi_i \quad \text{...... (2.30)}$$

$$D_i = n\left(1 - \frac{\phi_i}{\phi_{0i}}\right) = n\left(1 - \frac{1 - p_i}{1 - p_0}\right) \quad \text{...... (2.31)}$$

$$D_e = \sum D_i \quad \text{...... (2.32)}$$

Where, p_0 and p_i are porosities before and after the wave attack,

N_i is number of armor units in strip 'i',

a and b are strip height and strip width

ϕ_{0i} and ϕ_i are packing density before and after the wave attack,

n is times of strip width equivalent to cube size.

D_i and D_e are Dimensionless damage of each strip and equivalent damage

Fig. 2.5 Designed virtual net using Photoshop (Vanhoutte 2009)

2.6 BERM BREAKWATERS

2.6.1 Evolution of berm breakwaters

Berm breakwaters are different from the conventional breakwater with the presence of horizontal berm at or above SWL as shown in Fig. 2.6. The conventional breakwaters are designed in such a way that no damage or only little damage is allowed on the structure. This criterion necessitates the use of heavy and bulky rock or artificial concrete elements for armoring. The availability of large rock is seldom and even though if they are available transporting and placing them on the structure will be a problem which requires heavy equipments. The use of artificial armor unit as an alternative would also be a costly affair, since, the mould for casting and special equipments for lifting and placing costs come into picture. Thus, an alternative would be a safe and economical section where smaller size stones could be used.

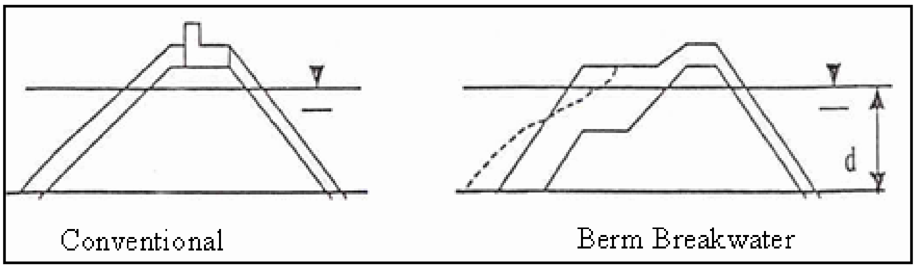


Fig. 2.6 Conventional vs. berm breakwaters

The berm breakwater which has an advantage of using lighter weight of the armor stones can be an alternative for the conventional rubble mound breakwater with heavy rocks or artificial armor units. The main concept of berm breakwater lies in the fact that the highly porous horizontal berm acts as a sacrificial member absorbing most of the wave energy thus reducing the impact of waves on the slope which in turn increases the stability of the structure.

Their natural response to hydrodynamic loads makes them economically attractive because, rock of lesser weight can be used in these types of breakwaters. The movement of material which results in sorting and nesting maximizes the inter-particle interlocking and the subsequent reforming of the profile that increases the stability of this structure (Van Gent 1993). The berm breakwater can be designed to optimize the utilization of the available quarry material for the available construction equipment. Berm breakwater performs better than the conventional structures when exposed to waves exceeding design conditions.

The berm breakwater concept is also relevant to many hot climate port locations where only relative small size rocks are available due to rock degradation. In many cases the total construction and repair costs are significantly lower for a berm breakwater compared to a conventional rubble mound breakwater, especially in hot climate locations and in large water depths (Andersen 2006).

A berm breakwater is considered to be a very rigid structure as it is very difficult to destroy, even a dynamically stable berm breakwater, by incoming head-on-waves, unless the structure is overtopped or the berm is too narrow. A conventional rubble mound breakwater fails easily and rapidly and repair operations are more difficult. The highly absorbing porous berm media and a flat slope around the water level gives very less reflection from a berm breakwater providing a better maneuvering condition at the entrance and less scour in front of the structure. Further, wave run-up and overtopping are smaller than that for a conventional straight and steeper breakwater slope.

One of the many advantages of berm breakwater is that the design is mainly supply based rather than demand based. The design of berm breakwater is governed by the available quarry yield in the field rather than computed quarry yield by a designer. The specifications should therefore be functional specifications, not demand specifications (CIRIA 2007).

The berm breakwaters may be divided into three categories:

- **Statically Stable Non- Reshaped:** In this condition few stones are allowed to move, similar to the condition for a conventional rubble mound breakwater. The $H/\Delta D$ value is less than 1.5 for this type of breakwater (PIANC 2003).
- **Statically Stable Reshaped:** In this condition the profile is allowed to reshape into a profile, which is stable and where the individual stones are also stable. The $H/\Delta D$ value ranges from 1.5-2.7 for a statically stable reshaped breakwater (PIANC 2003).
- **Dynamically Stable Reshaped:** In this condition, the profile is reshaped into a stable profile, but the individual stones may move up and down the front slope.

The H/ΔD value is more than 2.7 for a dynamically stable reshaped berm breakwater (PIANC 2003).

The first berm breakwaters built were with a homogeneous berm. Fig. 2.7 shows the cross section of a breakwater built in Racine, Michigan (Montgomery et al. 1988). This breakwater has a large berm in the front part of the breakwater, though the quarry stones are not very large. Such a design allows for berm deformation which will end up forming an equilibrium slope. Berm breakwaters like these have been built in North America, Europe, and other places, and many studies have been carried out on them (Baird and Hall 1984; Fournier et al. 1990; Burcharth and Frigaard 1987, 1988).

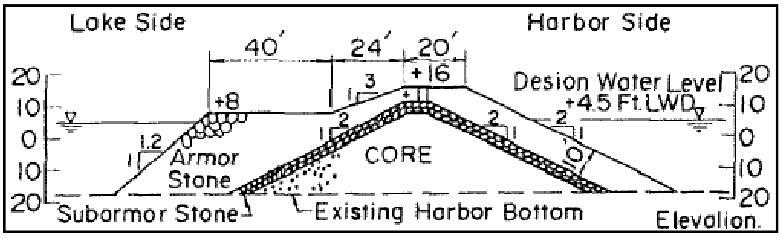


Fig. 2.7 Berm breakwater at Rachine, Michigan (Montgomery et al. 1988)

Along with the development of reshaping berm breakwater a new type of multilayered berm breakwater also came into existence in Iceland. This structure, named as Icelandic-type berm breakwater, consists of number of layers of different weights of armor placed systematically (Refer Fig. 2.8). Icelandic-type berm breakwater falls into either of the two categories: statically stable non-reshaped berm breakwater and statically stable reshaped berm breakwater.

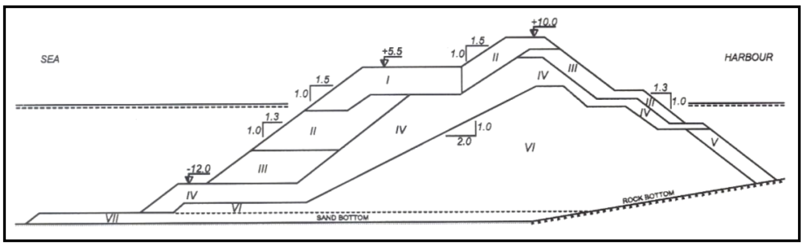


Fig. 2.8 Cross-section of the multi-layer berm breakwater under construction at Sirevag (Torum et al. 2003)

Various studies conducted on different types of berm breakwater are briefed in next few paragraphs.

Earlier studies (Popov 1961, Priest et al. 1964) have demonstrated that S-shape profiles are more stable than the original uniform slopes from which they develop. It was found that the stable profile after reshaping intersect the initial plane face at about 0.2 of the still water depth below the still water surface as shown in Fig. 2.9 (Priest et al. 1964).

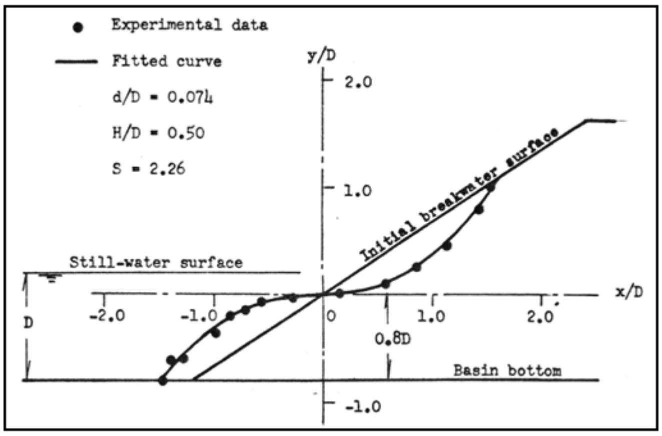


Fig. 2.9 Seaward profiles for steep, smooth waves acting upon a breakwater of concrete cube (Priest et al. 1964)

Bruun and Johannesson (1976) described the hydraulics of S-shaped breakwater and recommended the use of S-shaped breakwater geometry, instead of continuous single slope, for an increased safety and economy.

2.6.2 Stability of berm breakwaters

Naheer and Buslov (1983) from their studies concluded that the composite slope breakwater showed no improvement over the uniform slope, as far as wave run-up was concerned. They laid out the advantage of composite slope breakwater as the increase in the stability of the armor layer compared to uniform slope.

Torum et al. (1988) showed that berm breakwater is less sensitive to the variation in the significant wave height than the conventional breakwater. There was not a sudden collapse as normally is the case for a conventional breakwater. This is very useful for a location where there is a significant uncertainty in the wave climate.

Ergin et al. (1989) conducted a series of tests on three slope composite rubble mound breakwater, and concluded that under the same test conditions, same wave height, period range, water depth, and same armor stone, the three slope berm type section produce up to 90% less damage relative to the 1:2 single slope section. Also the increase in wave height of about 2.3 times the design wave height, the alternate section received damage ranging between 5 to 12% for the wave period tested.

Kao and Hall (1990) found that for armor stones having the same D_{50} values, the wave height is the single most important factor affecting the reshaping process of a dynamically stable breakwater. They also found that narrowly graded armor stones are less prone to reshaping and the effect of wave period and wave groupiness was less on berm width eroded and toe length after reshaping.

The mechanical strength (or quality) of rock has to be considered in prototype designs, especially for dynamically stable structures with large rock, as berm breakwaters. The quality of the rock is less important for small scale investigation and may not be considered (Van der Meer 1992).

Van Gent (1993) from his study on reshaping homogenous berm breakwaters with stone units concluded that rolling of armor units was the dominant failure mechanism compared to sliding, lifting and rocking.

Torum (1994) made an unsuccessful attempt to measure wave induced forces on armor units on berm breakwater using Morison type force method. It was found that forces on an armor unit were drag dominated.

The mobility of armor stones on berm breakwater was extensively studied by Lamberti et al. (1994), Lamberti and Tomasicchio (1997) and Archetti and Lamberti (1999). The research was performed within the range of $1.5 < H_o$ ($= N_s$) < 4.5. The main conclusions are summarized as follows:

- When $H_o = \sim 1.5 – 2$ the armor units on a berm breakwater attains mobility;
- When $2 < H_o < 3$ the mobility of stones is low;
- The mobility increases rapidly when $H_o > 3$;
- When $H_o = \sim 2.7$ a berm breakwater will attain a statically stable profile and when $H_o > \sim 2.7$ it reshapes itself into a dynamically stable profile.

Lissev and Torum (1996) from their studies concluded that the core of a berm breakwater can be extended into the berm. They suggested that with extending the core into the berm a cheaper berm breakwater structure can be obtained since the core material is cheaper than armor units.

Many researchers opined that waves of smaller wave steepness have highest wave force and increases berm reshaping to a larger extent (Archetti and Lamberti 1999 and Juhl and Sloth 1998).

From the studies of Torum on berm breakwater in shallow water Archetti and Lamberti (1999) concluded that the force of a wave in shallow water is less compared to deep water even for same significant wave height. This lesser force caused reduced recession in the structure for shallow water as compared to deep water.

Juhl and Sloth (1998) compared various profiles of berm breakwater (Fig. 2.10). They concluded that Profile 3 (an armor layer protecting both top and front of berm) was more effective than both Profile 4 and Profile 1 (hammerhead and thick layer at top of berm). They also concluded that by increasing armor stone size a reduction in berm width can be obtained but if fine materials (reduced permeability) are used in top of the berm or entire berm an increase in berm recession was observed. Also, with increased berm freeboard reshaping of berm was found to be decreasing even though there was an increase in berm volume.

Lissev and Daskalov (2000) carried out tests in the Hydraulic Research Laboratory at the University of Architecture, Civil Engineering with regard to extension of port of Burgas, the concept of berm type breakwater was offered as an alternative solution of conventional rubble mound breakwater covered by Tetrapods. They found that final profile of breakwater depended mainly on stone gradation.

Experiments on statically stable berm breakwaters have shown that increase in berm width reduces the damage to a large extent and for the same wave parameters the armor weight required for a berm breakwater can be reduced than that of a conventional breakwater (Hegde et al. 2002).

The PIANC (2003), according to the results of laboratory tests, reports that the recession of the berm (R_{ec}) for Icelandic type of berm breakwater structures is larger to some extent than for the homogeneous berm breakwater when the D_{n50} for the largest stone class is used to calculate H_oT_o and R_{ec} /D_{n50}.

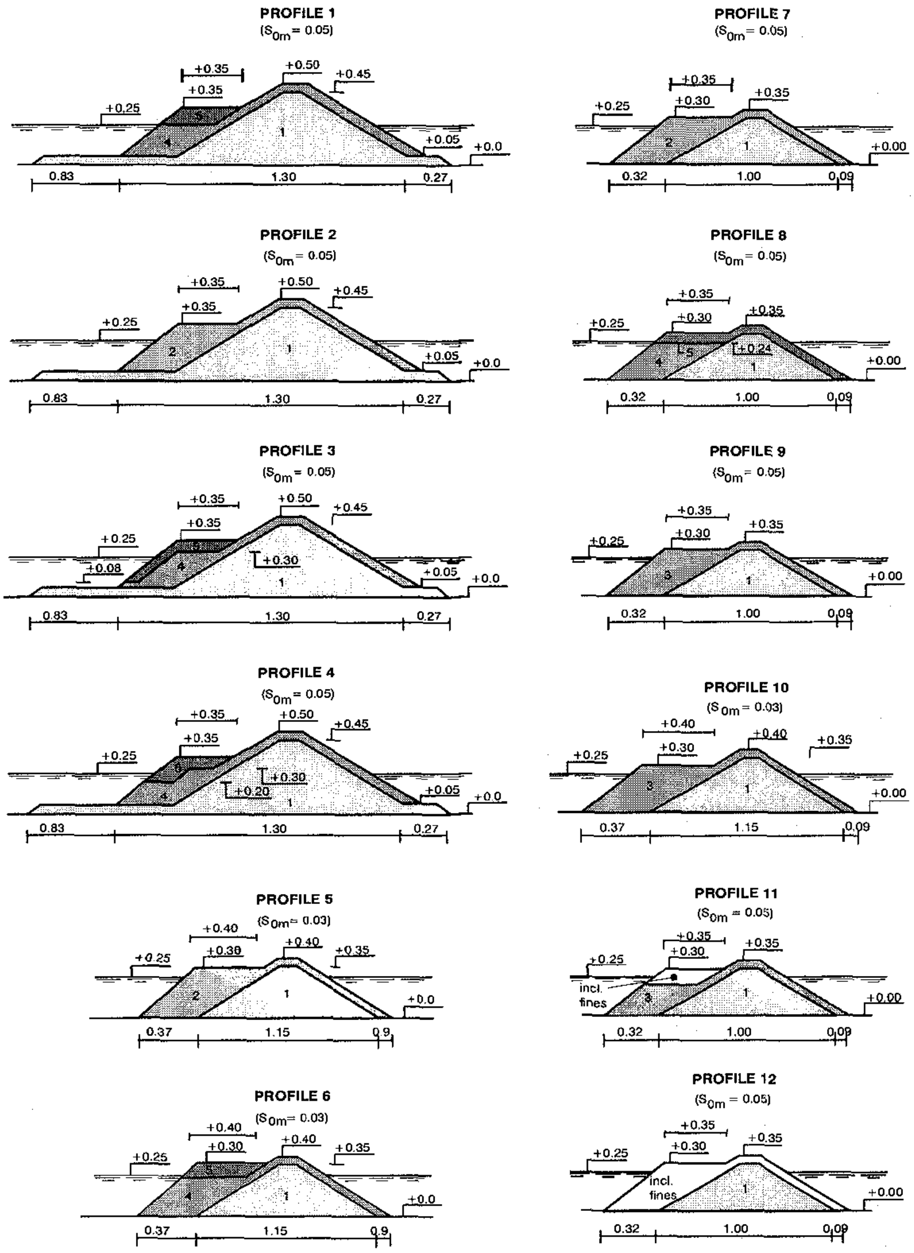


Fig. 2.10 Different profiles of berm breakwater considered for study (Juhl and Sloth 1998)

The most used parameters in relation to the stability of berm breakwaters are the following: (PIANC 2003)

$$N_s = H_o = \frac{H_s}{\Delta D_{n50}} \quad \text{...} \quad (2.33)$$

$$H_oT_o = \frac{H_s}{\Delta D_{n50}} T_z \sqrt{\frac{g}{D_{n50}}} \quad \text{...} (2.34)$$

Where, $\Delta = (\rho_s / \rho_w) - 1$.

H_s = Significant wave height,

$D_{n50} = (W_{50} / \rho_s)^{1/3}$

W_{50} = Median stone weight,

T_z = Mean wave period

ρ_s = Density of stone, ρ_w = Density of water.

Torum et al. (2003) have considered berm recession as one of the stability parameter for berm breakwaters expressed as R_{ec} / D_{n50}.

Tomasicchio et al. (2003) developed a theoretical model to evaluate the stone abrasion on reshaping armor slopes. The model forecasts the degradation of rock within the structure's lifetime and provides a reasonable prediction of volume loss caused by armor movements. But they neglected the effect of breakage, weathering and chemical agents on degradation of armor unit.

Rao et al. (2004) from their study on non-reshaping berm breakwaters found that increased berm width reduced damage to the structure. They found wave period had greater influence on stability of berm breakwaters. Increase in berm width was found to be ineffective in reducing wave run-up.

Fişkin (2004) studied the effect of armor stone gradation on stability of berm type breakwater and found that for a given berm width with increased reduction in stone size the cumulative local damage on the berm also increased. He also found that increased duration of wave attack did not affect the damage level of the structure. Also, when berm was placed above still water level resulted in lower wave run-up and water spray compared to models with berm below still water level.

Sigurdarson et al. (2006) in their survey mention that Icelandic type berm breakwater have been built for a design wave height of 7.5 m and further development to design for a wave height of 8 m is possible through various adjustments depending on the site conditions. The various adjustments may be use of high density stones or provide tandem breakwater or use of concrete armor units which reduces the reflection, overtopping as well as loading due to impact waves.

CEM (2006) states that even a damaged rubble mound berm can provide protection to primary armor layer and if the damage is severe then only it may be required to redesign the berm using larger units.

Recently study on prediction of stability of berm breakwater using artificial neural network was undertaken by Mandal et al. (2008). They have concluded that ANN models predicted the stability more accurately compared to empirical relationships and berm width is one major parameter affecting the stability. It was suggested that back-propagation neural network with Levenberg-Marquardt algorithm could be effectively used as advanced technique for predicting the stability of berm breakwater.

Dijkstra (2008) conducted exhaustive studies on stability of bermed slope breakwaters. He found that the berm length has a large influence on the stability of slopes and with the increase in berm length the stability also increases. It was found that two dominant forces affecting the stability *viz.* incident waves and the return of the current after reflection from the structure. The important role played by water level on these forces was highlighted. When water level is below berm level then incident forces will be prominent, when water level equals berm level both the forces will be in balance and when water level is above berm level the return current will be more prominent than the incident wave force. He also clearly specified that no design guidelines are available for a bermed slope breakwater and design is mainly dependent on past experience and model studies. The reason for not having design guidelines as told by him is the difficulty in modeling of complex processes related to the stability of the breakwater mainly the return current which has high impact on the stability of the structure.

Rao et al. (2008) presented the results of an experimental investigation on stability of non-reshaped berm breakwaters. From the work it was concluded that damage was relatively significant for shorter period waves in comparison with longer period waves, in the range of wave steepness from 0.008 to 0.043. The importance of water level was highlighted and it was found that the structure to be more stable when water depth above the berm was smaller. They also found mild slopes (1:2) being safer than steep slopes (1:1.5).

Moghim et al. (2011) carried out series of studies on homogenous berm breakwater to study the influence of various parameters on its stability. They observed that as wave height and wave period increases the berm recession also increases. They found most of the reshaping (90% of final reshaping) occurs within storm duration of 3000 waves. The demonstration by Andersen (2006) about the water depth in front of the structure was upheld by these researchers stating that larger water depths require larger deposition volume to dissipate the wave energy. Further, the effect of wave period was highlighted suggesting that the influence of wave period is less on the reshaping process.

Van Gent (2013) carried out tests on berm breakwater to develop a stability equation and to study influence of various berm parameters on its stability. It was found that when berm is below still water level (low berm) reduction in armor weight is less and the reduction increases as berm goes above still water level (high berm). It was observed damage was more near junction of berm and lower slope and in order to reduce this localized damage it was suggested to consider rounding off the transition near that junction rather than providing a sharp transition, in order to reduce the exposure of armor units.

Shekari and Shafieefar (2013) conducted experimental studies on reshaping berm breakwaters under irregular wave attack. They also have concluded that with increase in berm width and higher berm elevation above still water level reduces the berm recession. They have proposed a formula for estimating berm recession considering various parameters.

2.7 DAMAGE OF BERM BREAKWATERS

2.7.1 Statically stable non-reshaped berm breakwaters

The damage of a statically stable non-reshaped berm breakwater is similar to that of a rubble mound breakwater which has been dealt in detail in Section 2.5.

2.7.2 Statically stable reshaped berm breakwaters

In case of statically stable reshaped berm breakwater, the berm recession and exposure of underlayer/ secondary layer can be adopted for measuring the damage to the structure. Since, the structure is allowed to reshape up to a certain extent the damage parameters explained earlier for a rubble mound breakwater cannot be applied. Davies et al. (1994) during their studies on damage prediction of rubble mound breakwaters have given specific reasons for not adopting damage level ‘S’ in case of reshaped as well as reshaping berm breakwaters. They have quoted that usage of damage level ‘S’ would loose its relevance since the berm breakwaters are allowed to reshape. In order to substantiate their statement, they have explained the damage level ‘S’ considering different thickness of primary layers as shown in Fig. 2.10. A value of S = 3, as in the Fig. 2.11, could correspond to a deep hole in the armour layer, or it could represent shallow damage spread over a wider area. So, the damage level ‘S’ depends mainly on size of the armor and armor layer thickness.

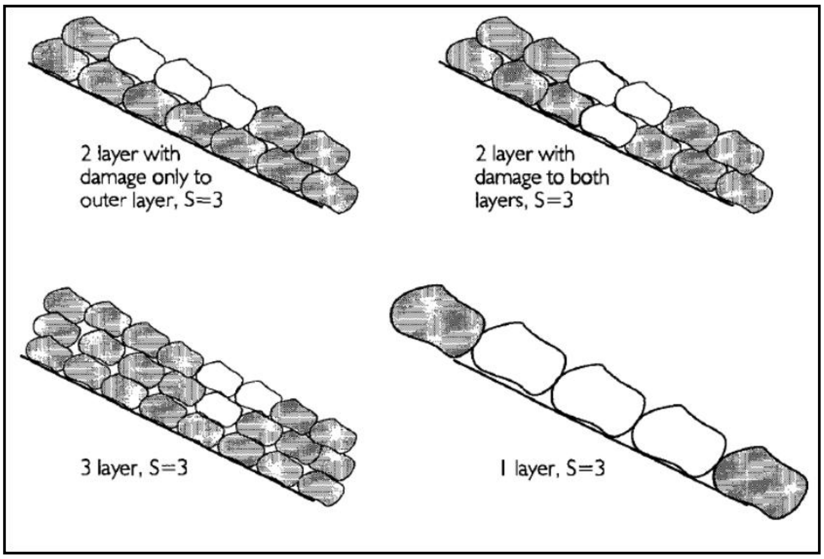


Fig. 2.11 Damage level ‘S’ for different thickness of primary layer (Davies et al. 1994)

In order to overcome the above disadvantage they have suggested measuring the depth of cover layer (d_c) remaining after the wave attack and the minimum depth of cover at any point of time must not be less than one ‘D_{n50}’. If the value of ‘d_c’ drops to zero then the damage is considered severe with the exposure of underlayer.

2.7.2.1 Berm recession

An important feature of reshaping is the recession of the berm as shown in Fig. 2.12. Recession (R_{ec}) is the erosion of horizontal berm due to the attack of wave. Hall and Kao (1991) investigated the influence of stones on the reshaping of the berm. They arrived at an equation for the recession of a homogenous berm. Torum (1998) gave a dimensionless recession equation, R_{ec}/D_{n50}, which was updated by Torum et al. (1999) when the waves are approaching the breakwater is almost normal to the breakwater longitudinal axis as:

$$\frac{R_{ec}}{D_{n50}} = 0.0000027(H_oT_o)^3 + 0.000009(H_oT_o)^2 + 0.11(H_oT_o) - 0.8 \text{ .... (2.35)}$$

Where $H_oT_o > 20\text{-}30$

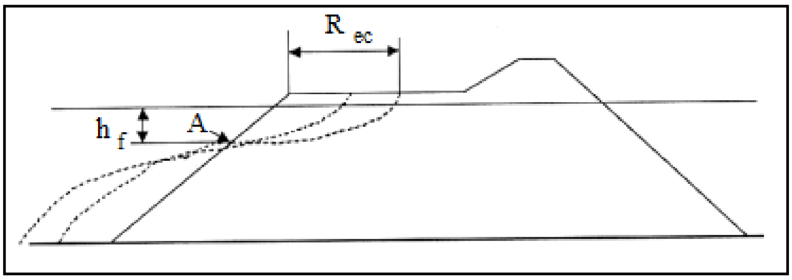


Fig. 2.12 Definition of Recession (R_{ec})

Sayao (1999) defines the stability of berm breakwaters as follows, "A stable, dynamic, berm breakwater is a structure with $W_r/B \approx 1$ for the design storm condition, where W_r = reshaped portion of the berm width, B = berm width (Refer Fig. 2.13). Thus, if during testing $W_r/B \leq 1$, the berm is considered as stable. If the berm reshapes more than its design width at berm elevation ($W_r/B > 1$), then the berm is considered unstable and new tests are performed with an increased B.

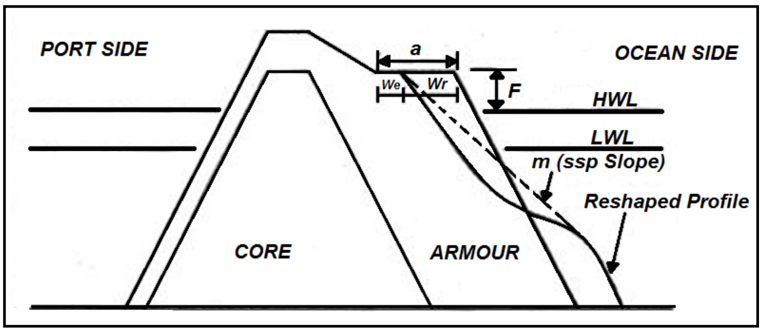


Fig. 2.13 Berm parameters (Sayao 1999)

As quoted by Torum et al. (2003), Menze (2000) added some terms to the Eq. 2.35 to take into account the gradation of the stones and the water depth. The recession equation arrived at is then:

$$\frac{R_{ec}}{D_{n50}} = 0.0000027(H_o T_o)^3 + 0.000009(H_o T_o)^2 + 0.11(H_o T_o) - f_g - f_d \text{ .... (2.36)}$$

Where, Gradation factor, $f_g = -9.91\left(\frac{D_{n85}}{D_{n15}}\right)^2 + 23.9\left(\frac{D_{n85}}{D_{n15}}\right) - 10.5$ and is valid for $1.3 < D_{n85}/D_{n15} < 1.8$.,

Depth factor, $f_d = -0.16\left(\frac{d}{D_{n50}}\right) + 4.0$, within the range $12.5 < d/D_{n50} < 25$

Referring to Fig. 2.11, as an approximation, h_f can be obtained as (PIANC 2003):

$$\frac{h_f}{D_{n50}} = 0.2\left(\frac{d}{D_{n50}}\right) + 0.5, \text{ within the range } 10 < d/D_{n50} < 25 \text{ .... (2.37)}$$

Torum et al. (2003) conducted laboratory tests for multilayer berm breakwaters and calculated berm recession using equation given by Menze (2000) for homogenous berm breakwater. Fig. 2.14 shows the dimensionless recession relation v/s H_oT_o for multilayer berm breakwaters. Torum et al. (2003) concluded that till then no formula has been developed for calculating berm recession of a multilayered berm breakwater and hence the equation by Menze (2000) can be considered for feasibility studies. Further, it was concluded that the smaller stone size is sufficient for large density stones compared to small density stones, provided the same absolute recession is considered.

Sigurdarson et al. (2007) came up with a more simple formula for predicting berm recession for an Icelandic type of berm breakwater wherein they assumed that the influence of stone grading and water depth on berm recession is small.

$$\frac{R_{ec}}{D_{n50}} = 0.037(H_o T_o - S_c)^{1.34} \text{ .... (2.38)}$$

Where, S_c is the scatter in recession measurements and $R_{ec}/D_{n50} = 0$ for $H_oT_o < S_c$.

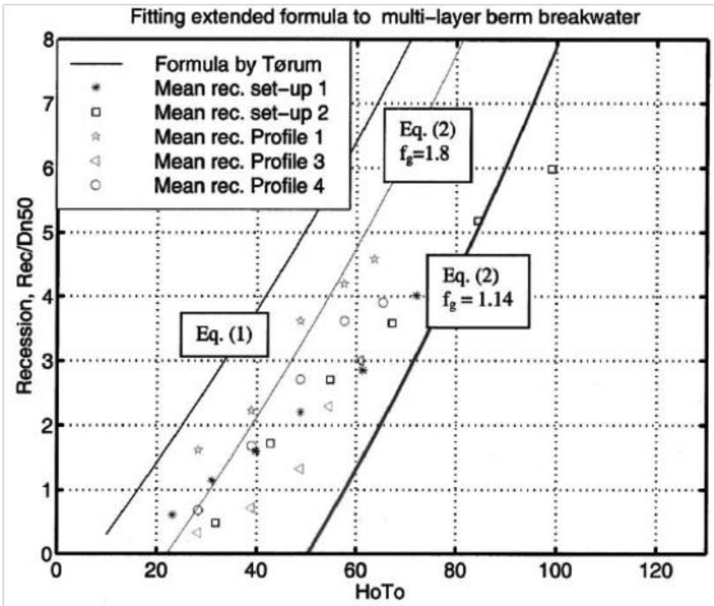


Fig. 2.14 Recession of a multi layer berm breakwater (Torum et al. 2003)

Rao et al. (2010) from their study in reshaping berm breakwater concluded that berm recession is influenced largely by wave period. Also, they found that with the increase in equivalent surf similarity parameter and decrease in wave steepness, the berm recession was found to decrease.

Andersen and Burcharth (2010) developed an equation for calculating berm recession for a homogenous berm breakwater. They suggested that the equation can also be adopted where the core is extended into the berm as long as core level is below the still water level. They found that 'R_{ec}' and 'h_f' are the only good measures for reshaped profile when berm is above still water level and when berm is below still water level the damage to the crest was high even with recession being zero. Hence, recession cannot be considered as damage parameter when berm level is below still water level.

$$\frac{R_{ec}}{D_{n50}} = f_{hb}\left[f_{Ho}\,\frac{2.2(h-1.2)h_s}{h-h_b}\cdot f_\beta \cdot f_N \cdot f_{grading}\cdot f_{skewness} - \frac{[\cot(\alpha_d)-1.05]}{2.D_{n50}}.[h-h_b]\right] \quad \text{……… (2.39)}$$

Where,

$f_\beta = \cos\beta$

$$h_s = 0.65 H_{mo} s_{om}^{-0.3} f_N f_\beta \quad \text{and} \quad s_{om} = \frac{H_{mo}}{\frac{g}{2\pi}.T_{01}^2}$$

$$f_{hb} = \begin{cases} 1 & for \ \dfrac{h_b}{h_{mo}} \le 0.1 \\ 1.18.\exp\left(-1.64.\dfrac{h_b}{h_{mo}}\right) & for \ \dfrac{h_b}{h_{mo}} > 0.1 \end{cases}$$

$$f_N = \begin{cases} \left(\dfrac{N}{3000}\right)^{-0.046 H_o + 0.3} & for H_o < 5 \\ \left(\dfrac{N}{3000}\right)^{0.07} & for H_o > 5 \end{cases}$$

$$H_o = \frac{H_{mo}}{\Delta D_{n50}} \quad \text{and} \quad T_o = \sqrt{\frac{g}{D_{n50}}} T_{01}$$

$$T_o^* = \frac{19.8.\exp\left(-\frac{7.08}{H_o}\right).s_{om}^{-0.5} - 10.5}{0.05 H_o}$$

$$f_{Ho} = \begin{cases} 19.8.\exp\left(-\frac{7.08}{H_o}\right).s_{om}^{-0.5} & for T_o \ge T_o^* \\ 0.05 H_o T_o + 10.5 & for T_o < T_o^* \end{cases}$$

$$f_{skewness} = \exp\left(1.5 b_1^2\right)$$

$$f_{grading} = \begin{cases} 1 & for \quad f_g \le 1.5 \\ 0.43 f_g + 0.355 & for \quad 1.5 < f_g < 2.5 \\ 1.43 & for \quad f_g \ge 2.5 \end{cases}$$

In the absence of wave skewness information the surface skewness can be predicted from the Ursell number using:

$$b_1 = 0.54 Ur^{0.54}$$

$$Ur = \frac{H_{mo}}{2.h.\left(k_p h\right)^2} \quad \text{and} \quad k_p = \frac{2\pi}{L_p}$$

The berm recession of Icelandic type of berm breakwaters cannot be defined by earlier definitions as suggested by Sigurdarson and Van der Meer (2011). They gave a more refined definition stating that instead of measuring the recession on top of berm it can be measured either as the mean or the maximum horizontal recession on the front slope of the berm. Two new terms 'R_{ecmax}' and 'R_{ecav}' were coined by them to measure the recession. R_{ecmax}, the maximum recession distance, is the greatest measured recession on any individual profile and R_{ecav}, the average recession distance, is the recession of the average profile averaged between low water level and top of the berm. They found that initial recession is largely dependent on wave height rather than wave period and gave an empirical relation based on this condition as,

$$R_{ec}/D_{n50} = 2.5\left(\left(H_s/\Delta D_{n50}\right) - S_c\right)^2 \qquad (2.40)$$

With $R_{ec}/D_{n50} = 0$ for $H_s/\Delta D_{n50} < S_c$ and $H_s/\Delta D_{n50} < 2.5$

Moghim et al. (2011) introduced a new term $H_o\sqrt{T_o}$ instead of H_oT_o which includes some sea state and structural parameters. The influence of different parameters on berm recession was first studied individually and then the important influencing parameters were considered for developing the equation. The equation was then developed by curve fitting and nonlinear regression analysis considering the important influencing parameters. The recession estimated using this formula was found to have a better correlation with experimental data of their study as well as data of other experiments within the range specified. The dimensionless recession is given by,

$$\frac{R_{ec}}{D_{n50}} = \left(10.4\left(H_o\sqrt{T_o}\right)^{0.14} - 13.6\right)\left\{1.61 - \exp\left[-2.2\left(\frac{N}{3000}\right)\right]\right\}\left(\frac{h_{br}}{H_s}\right)^{-0.2}\left(\frac{d}{D_{n50}}\right)^{0.56}$$

$$\text{for } H_o\sqrt{T_o} < 17 \qquad (2.41)$$

$$\frac{R_{ec}}{D_{n50}} = \left(0.089\left(H_o\sqrt{T_o}\right) + 0.49\right)\left\{1.61 - \exp\left[-2.2\left(\frac{N}{3000}\right)\right]\right\}\left(\frac{h_{br}}{H_s}\right)^{-0.2}\left(\frac{d}{D_{n50}}\right)^{0.56}$$

$$\text{for } H_o\sqrt{T_o} \geq 17 \qquad (2.42)$$

Within the range of,

$7.7 < H_o\sqrt{T_o} < 24.4$,	$500 < N < 6000$,	$8.0 < d/D_{n50} < 16.5$
$1.2 < f_g < 1.5$,	$0.09 < d/L < 0.25$,	$0.12 < h_{br}/H_s < 1.24$

Shekari and Shafieefar (2013) have studied the effect of irregular waves on homogenous reshaping berm breakwater. They observed that increase in berm elevation above still water level decreases berm recession. Further, they have proposed an equation for the estimation of berm recession considering various parameters affecting the stability of breakwater.

$$\frac{R_{ec}}{D_{n50}} = \left(-0.016\left(H_o\sqrt{T_o}\right)^2 + 1.59H_o\sqrt{T_o} - 9.86\right)\left\{1.72 - \exp\left[-2.19\left(\frac{N}{3000}\right)\right]\right\}$$
$$\left(\frac{B}{D_{n50}}\right)^{-0.15}\left(\frac{h_b}{H_s}\right)^{-0.21} \quad \ldots (2.43)$$

The Eq. 2.43 is valid within the following conditions

$7.09 < H_o\sqrt{T_o} < 23.52$
$500 < N < 6000$
$0.22 < h_b/H_s < 1.57$
$9.6 < d/D_{n50} < 14.11$
$14 < B/D_{n50} < 29.41$

2.7.3 Dynamically stable berm breakwater

For a dynamically stable berm breakwater the armor units are allowed to move in its axis even after the structure has attained equilibrium shape. Hence, the damage in this case is quantified based on its profile development and berm recession. The berm recession has already been explained in the previous Section 2.7.2.1.

2.7.3.1 Profile development

The dynamic stability is defined by the formation of a profile which can deviate substantially from the initial profile. In order to describe a dynamically stable profile the profile has to be schematized into profile parameter.

Van der Meer and Pilarczyk (1986) developed a model for predicting profile development of dynamically stable rock slopes. The developed model predicted the profiles of slopes with any arbitrary shape under varying wave conditions. The model also estimated the length of the gentle part of a dynamically stable berm breakwater and also its position part below the still water level.

Van der Meer (1988a, 1992) developed a computational model, based on empirical equations, which can be used to predict profile changes. The model was developed by combining a numerical description of both wave loading and the particulate structure of breakwater armor layer. The model was then used to develop computational computer program BREAKWAT. Fig. 2.15 shows a schematized model for a dynamically profile development on a 1:1.5 uniform slope. Various parameters were considered for developing a schematized profile. Another model was developed by Norton and Holmes (1992) on the same lines of Van der Meer (1990), but considering individual displacements of stones under monochromatic wave attack. The model provided a good qualitative prediction of profile development. Boundary conditions for Van der Meer model were,

- Applicable within the range of $H_s/\Delta D_{n50} = 3 - 500$.
- Arbitrary initial slope is essential.
- Crest has to be above still water level.
- Computation of profiles for a sequence of storms is by using previously computed profile as initial profile.

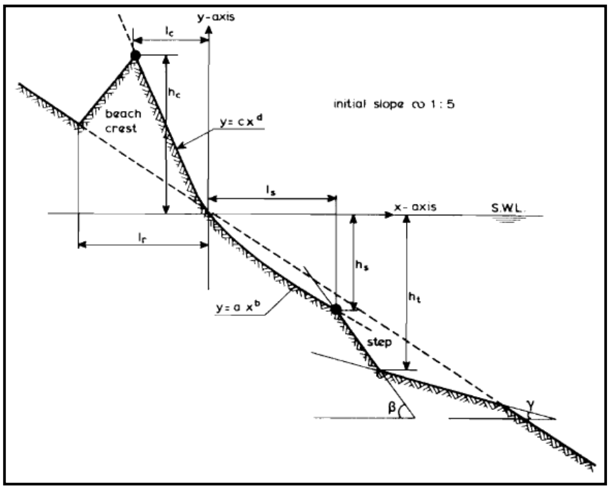


Fig. 2.15 Schematized profile of 1:5 initial slope (Van der Meer 1992)

Van Gent (1993) from the physical model and numerical studies concluded that as wave height and period increases, the reshaped profile also lengthens. It was found that smaller stone units lead to more accretion below still water level. Also, initial slope hardly influenced reshaping near still water level but further upward or downward the reshaped profiles evolve towards initial slope. Van Gent (1996) developed a numerical model for predicting the profile of a dynamically stable breakwater. He considered drag force, inertia force and lift force for modeling.

Kortenhaus et al. (2004) suggests using Van der Meer method (1992) for computing the reshaped profiles of a dynamically stable breakwater. A good agreement was found between the measured and calculated profiles. But, for a statically stable reshaping breakwater the damage was over predicted by the same method.

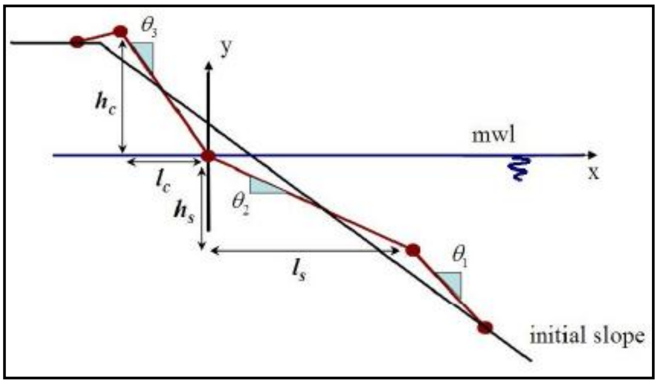


Fig. 2.16 Parameterization of a generic reshaped profile (Merli 2009)

Merli (2009) developed a simple numerical model for calculation of reshaped profile. In his study various parameters were considered for schematizing the reshaped profile as shown in Fig. 2.16. He concluded from his studies that more graded wide material is more impermeable, thus reducing the dissipation of wave energy and enhancing the instability.

2.8 WAVE RUN-UP AND WAVE RUN-DOWN IN BERM BREAKWATERS

Many researchers have concluded that a low berm is very efficient from the run-up point of view (Torum et al. 1988). As quoted by PIANC (2003), Pilarczyk found that run-up on slope protection decreases with increasing berm width and the reduction rapidly falls off once a certain minimum width is exceeded: for $B > 0.25\ L_o$ for non breaking waves and for $B > 4\ H_s$ for strong breaking waves i.e., for $H_s / L_o > 0.03$.

Ahrens and Ward (1991) observed that reduction in run-up as a result of the berm is a rather modest 20%. In order to estimate the maximum wave run-up on a revetment fronted by a rubble berm they suggested an empirical equation.

De Waal and Van der Meer (1992) from their studies found that for a berm at SWL there was large reduction in run-up. They also found that the influence of the berm on run-up is negligible when the berm is about 1.5 H_s above or below SWL. They introduced a new term 'ξ_{eq}' called equivalent surf similarity parameter calculated considering an equivalent slope (Fig. 2.17). They gave a prediction equation for run-up considering the influence of roughness, shallow water effect and other parameters as,

$$\frac{R_{u2\%}}{H_s} = \begin{cases} 1.5\gamma_f \gamma_h \gamma_\beta \xi_{p,eq} & 0.5 < \xi_{eq} \le 2 \\ 3.0\gamma_f \gamma_h \gamma_\beta & \xi_{eq} > 2 \end{cases} \quad \text{.....................................} \ (2.44)$$

Where, γ_f = influence factor for roughness

γ_h = influence factor for shallow water

γ_β = influence factor for oblique wave attack

$\xi_{p,eq}$ = breaker parameter based on an equivalent slope = $\gamma_b \, \xi_p$

γ_b = influence factor for berm

γ_f = 0.60 for rough slope due to rubble layer of berm

$$\gamma_h = \frac{H_{2\%}}{1.4H_s}$$

For short crested waves, $\gamma_\beta = 1 - 0.0022\beta$

For long crested waves, $\gamma_\beta = \begin{cases} 1.0 & for \quad 0^o \le \beta \le 10^o \\ \cos(\beta - 10^o) & for \quad 10^o < \beta \le 63^o \\ 0.6 & for \quad \beta > 63^o \end{cases}$

$\gamma_b = 1 - r_B\,(1 - r_{dB})$ with $0.6 \le \gamma_b \le 1.0$

Where, r_B = reduction of average slope ($\tan\alpha$) caused by the berm width B.

r_{dB} = reduction of the influence of a berm caused by the berm depth d_B (a berm at SWL yields $r_{dB} = 0$)

$$r_B = \frac{B/H_B}{2\cot\alpha + B/H_B} \quad \text{and}$$

$$r_{dB} = \tfrac{1}{2}\left(\frac{d_B}{H_s}\right)^2 \qquad \text{with } 0 \le r_{dB} \le 1$$

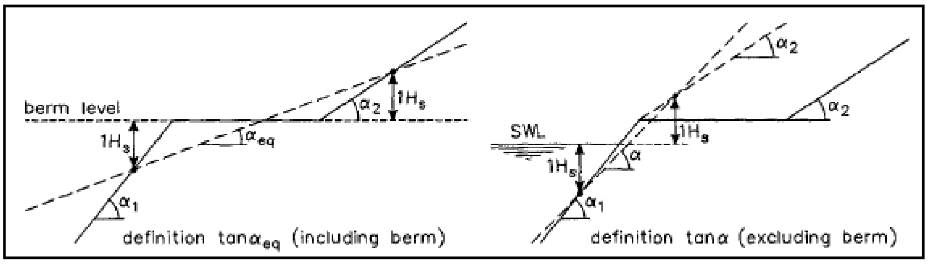


Fig. 2.17 Definition of equivalent and average slope

(De waal and Van der Meer 1992)

For a structure without berm 'r_B' is equal to 0 and when berm is at SWL 'r_{dB}' is 0.

The influence factor γ_b has a lower limit of 0.6. This implies that in a situation where γ_b is 0.6 an increase of berm width will not lead to a further reduction of run-up. They also found optimum berm width when berm is at SWL and γ_b =0.6 as,

$$B = \frac{4}{3} H_s \cot\alpha \qquad (2.45)$$

Van der Meer (1998) has proposed the following design formula for estimating wave run-up due to irregular waves.

$$R_{u2\%} / H_s = \max(1.6\, \xi_{op}\, \gamma \,,\, 3.2\, \gamma_f\, \gamma_\beta) \text{ in the range } 0.5 < \gamma_b\, \xi_{op} < 4 \text{ or } 5 \qquad (2.46)$$

where, $R_{u2\%}$ = Run-up level measured vertically above SWL, which is exceeded by two percent of incoming waves.

γ = Reduction factor = $\gamma_b\, \gamma_f\, \gamma_\beta$

γ_b = Represents the effect of the presence of the berm

γ_f = Represents the effect of rough surface

γ_β = Represents the effect of wave obliquity.

Rao et al. (2004) found that berm width has no influence on wave run-up. The wave run-up was found to increase with decrease in armor weight from W_o/W = 1 to W_o/W = 0.9 but for W_o/W = 0.7 the wave run-up remained almost same as that for armor weight W_o/W = 0.9. This shows that reduction in weight of armor has influence on wave run-up upto certain extent after which it does not influence it.

Fişkin (2004) found from his study concluded that when berm was constructed above Still Water Level (SWL), energy dissipation of the waves travelling along the width of the berm was increased due to the pores between the armor stones forming the berm. This resulted in lower wave run-up and water spray when compared with the model investigations done on the berm being below SWL.

Rao et al. (2007) from their study on berm breakwater with reduced armor weight have found that the maximum run-up and run-down are respectively 1.28 times and 1.07 times the deep water wave height. Rao et al. (2008) have found that the slope of breakwater has a lot of influence on wave run-up and with the change of slope from 1:1.5 to 1:2 the run-up was reduced by 36%. Also with the increase in wave steepness, the wave run-up was found to decrease. They found that wave run-down was more significant than the wave run-up when SWL was above the berm and run-down was less significant than run-up when SWL was below the berm. Studies on statically stable berm breakwater by Rao et al. (2010) have concluded that the relative wave run-up and run-down was less when berm level is equal to still water level.

Van Broekhoven (2011) in his study on influence of armor layer and permeability on run-up for two different slopes of 1:1.5 and 1:2 found that for (regular) surging waves ($\xi > 4.0$) the surface roughness has a negligible influence on the reduction of the wave run-up. For $\xi < 4.0$ both the surface roughness and the permeability of the armor layer influences the wave run-up. Also, for values of $D_{n50}/H < 0.7$ the stone diameter has no influence on the reduction of the relative wave run-up. For values of D_{n50}/H between 0.7 – 1.0 a slight difference is visible, and for $D_{n50}/H > 1.0$ large influence of diameter of stone can be found on wave run-up.

2.9 DESIGN ASPECTS OF BERM BREAKWATER

Different countries follow different design procedure of berm breakwaters. There are no specific design equation or comprehensive design criteria sets for berm breakwaters similar to design equations for conventional breakwaters.

Many aspects are to be considered when designing a berm breakwater in addition to environmental conditions, quarry yield and stone breaking strength. For the final design the following aspects may also be taken into account:

- Reshaping
- Lateral transport of stones
- Soil stability
- Wave overtopping
- Construction methods
- Scour and scour protection
- Cost evaluation.

Torum et al. (1999) introduced the limit state design for berm breakwaters and have discussed various limit states. They have also given a relation between stone velocity and modified stability number studying previous experimental results of MAST II project. Preliminary studies on rock strength (drop test) were also conducted.

The stone velocity is given by,

$$V_{stones} = \sqrt{H_s g}.\alpha.\left(1-\left(\frac{N_o^{**}/2}{N_s^{**}}\right)^{\gamma}\right)^{\delta} \quad \text{...} \text{ (2.47)}$$

Where,

$$N_s^{**} = \frac{H_k}{C_k \Delta D_{n50}}\left(\frac{s_{mo}}{s_{mk}}\right)^{-(1/5)} \left(\cos\beta_o\right)^{2/5} \text{ (Lamberti and Tomasicchio 1997)}$$

s_{mk} = reference wave steepness = 0.03

s_{mo} = actual wave steepness

β_o = angle between breakwater axis and wave direction

$H_k/C_k = H_s$ for deep water

N_o^{**} = value of N_s^{**} at start of reshaping of berm breakwater

The best estimates for α, γ and β are given in Table 2.7 assuming fixed values of parameters γ and N_o^{**}.

Table 2.7 Best parameter estimates for velocity statistics (Torum et al. 1999)

Fixed paramaeters, γ, N_o^{**}	**Best estimates**			
	N_o^{**}	**α**	**β**	**γ**
V_{median}	2.0	0.064	0.5	0
V_{mean}	2.0	0.11	0.5	0.5
V_{max}	2.0	3.87	0.5	3

Limit state design on hydraulic stability

The limit state design is an attempt to establish a rational bridge between the three design philosophies as such “non-reshaped statically stable, reshaping statically stable or dynamically stable”, by considering the actual block degradation resistance as per Torum et al. (1999). The different limit states that are used in Norway are:

Serviceability Limit States (SLS): The Serviceability Limit States (SLS) is applied to know the general functional requirements with the change in profile before and after reshaping. Also, this limit state prohibits the movement of armor blocks under normal sea states other than severe sea states.

This is to be checked for sea states occurring 50 times during the design lifetime, requirement:

- No significant motions of the stones due to waves
- No wave transformation through the breakwater.

Ultimate Limit States (ULS): The Ultimate Limit State (ULS) is adopted to check the reshaping of breakwater based on the quality of rock.

This is checked for a sea state with a 100 year recurrence period, requirement:

- It is acceptable for the berm to reshape. However, the residual berm width should not be less than $4XD_{n50}$. After reshaping the distance from the reshaped profile to the lower layer with smaller stones, possibly a filter layer should be larger than $1.5D_{n50}$ or at least 2m. The armor stones should be able to withstand the reshaping without splitting, which reduces D_{n50} due to the motion of the stones.

Fatigue Limit States (FLS): This limit state is utilized to check the accumulated reshaping after all large storms and also the block degradation.

This is usually verified for a sea state of 10 year recurrence period after reshaping in ULS, requirement:

- No significant further reshaping must be allowed. Also, additional splitting and abrasion of stones must not be allowed.

Accidental Limit States (ALS): This limit state is adopted to check the structural integrity for the worst credible sea state. It makes sure of the necessary safety and toughness margins of a breakwater structure.

To be checked for a sea state of 10,000 year recurrence period after reshaping in ULS, requirement:

- The breakwater must be intact even after wave attack.

Van Gent (1993) suggests that erosion of berm to be such that no core has to be exposed. The primary layer above core in any case must be at least two to three stone diameters thick.

Van der Meer (1998) gave a method to calculate first approximate berm width. As quoted by him “First draw a straight line from the crest of the structure to the filter layer at the bottom under a slope of 1:4. This gives more or less the total amount of rock that is required at the seaward side. Then draw a steep upper slope (around 1:1.5) until the desired berm level. Redistribution of the required amount of rock will give a first estimation of the berm width.” This is illustrated in Fig. 2.18.

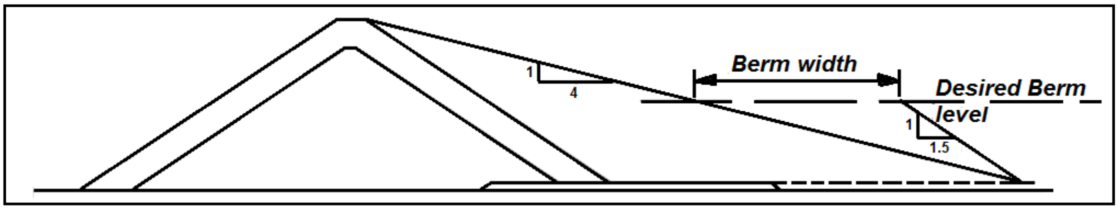


Fig. 2.18 First estimation of berm width as suggested by Van der Meer (1998)

For practical purpose the berm width can be calculated using equation suggested by (Fişkin 2004).

$$B_b = D_{n50}\left[-10.4 + 0.5\left(\frac{H_s}{\Delta D_{n50}}\right)^{2.5} - 1.1\left(\frac{D_{n85}}{D_{n15}}\right)^2 + 7.5\left(\frac{D_{n85}}{D_{n15}}\right) + 6.1P_r\right] \text{............ (2.48)}$$

Where, B_b = berm width

P_r = fraction of rounded stones

Berm width could be designed between the values as B_b to B=L/4 (Fişkin 2004).

CIRIA (2007) have given some design guidelines for initial dimensions of a berm breakwater. The berm height (h_b) can be taken as 0.5 to 0.9 times H_s above design water level. The berm height has no significant effect on berm recession within the range h_b/D_{n50} = 2 – 4 (Torum 1998). The minimum berm width must be equal to berm recession due to maximum design waves.

Sigurdarson et al. (2007) listed the design guidelines developed for an Icelandic type of berm breakwater as:

- The upper layer of the berm must contain two layers of rock and it will extend on the down slope to at least mean sea level;
- The rock size of this layer is determined by H_o = 2.0. Larger rock may be used too;
- Slope of 1:1.5 can be maintained above and below the berm;
- The berm width can be taken equal to 2.5 - 3.0 H_S;
- The berm level preferably must be 0.65H_S above design water level;

- The crest height can be obtained by $R_C/H_S * s_{op}^{1/3} = 0.35$;

Van Gent (2013) has developed stability equation for berm breakwater focusing the slope above berm and slope below berm separately. The equation is:

For upper slope:

$$S = \gamma_{berm}\gamma_{pos}\left(\frac{1}{c_{plunging}}\frac{H_s}{\Delta D_{n50}}\xi_m^{0.5}P^{-0.18}N^{0.1}\frac{H_{2\%}}{H_s}\right)^5 \quad \text{.....................................} (2.49)$$

With

$$\gamma_{pos} = \left(0.25 + 0.125\cot\alpha\left(1+\frac{h_b}{H_s}\right)\right)^5 \quad \text{for } 0 \le \gamma_{pos} \le 1$$

$$\gamma_{berm} = 1 - 0.15\xi_m\left(\frac{B}{H_s}\right)^{0.3} \qquad \text{for } \gamma_{berm} \ge 0$$

For lower slope:

$$S = \gamma_{berm}\left(\frac{1}{c_{plunging}}\frac{H_s}{\Delta D_{n50}}\xi_m^{0.5}P^{-0.18}N^{0.1}\frac{H_{2\%}}{H_s}\right)^5$$

With

$$\gamma_{berm} = 1 - 0.12\left(\frac{B}{H_s}\right)\left(\frac{h_b}{H_s}\right) \qquad \text{for } h_b/H_s \ge 0$$

$$\gamma_{berm} = 1 - 0.02\xi_m\left(\frac{B}{H_s}\right)\left|\frac{h_b}{H_s}\right| \qquad \text{for } h_b/H_s \le 0$$

The equation can be applied under the following parametric conditions.

Parameter	**Symbol**	**Value**
Slope angle	$\cot\alpha$	2 and 4
Relative density	Δ	1.7
Wave steepness	s_p	0.015 – 0.040
Berm level w.r.t SWL (m)	h_b	- 0.1 – 0.1
Grading armor material	D_{n85}/D_{n15}	1.5
Berm width	B	0.0 – 0.8
Number of waves	N	1000
Stability parameter	$H_s/\Delta D_{n50}$	2 – 8
Damage level	S	< 20

2.10 ECONOMICAL DESIGN OF BERM BREAKWATERS

Baird and Hall (1984) quoted that by building berm breakwaters using smaller stones and utilizing quarry yield economically, there can be 50% to 70% cost savings for a site compared to traditional breakwater.

Berm breakwater can be built with armor stones which are 20% to 30% lower in weight than that required for an equivalent traditional uniform sloped breakwater (Gadre et al. 1991).

Van Gent (1993) also has quoted that berm breakwater structure requires stone weight two to ten times less than that required by conventional breakwater.

Hauer et al. (1995) performed a detailed cost comparison analysis between a conventional and berm breakwater. For a specific harbor they concluded that the cost of conventional breakwater under all considered circumstances was more than berm breakwater. Also, deviation in quarry yield may result in more than 100% growth in total costs of a conventional breakwater whereas berm breakwater would be only slightly affected.

Different types of breakwaters were considered for cost comparison by Tutuarima and D'Angremond (1998). From their results it was found that even though berm breakwaters required large volume of rocks they yield low costs mainly because of higher efficiency in quarry utilization.

Hegde et al. (2002) have concluded that a more economic section could be a structure with smaller armor unit, where profile development being allowed in order to reach a stable profile.

The comparison of hypothetical conventional and berm breakwater by CIRIA (2007) showed that conventional breakwater required 36% extra cost than that required for berm breakwater.

Andersen and Burcharth (2010) have suggested that the added stability of multilayer berm breakwater (Icelandic berm breakwater) should be compared to the extra cost of the more sorting and the more complicated construction method.

2.11 STUDIES ON ARTIFICIAL ARMOR UNITS

The stability of some artificial units under random wave was studied by Van der Meer (1988b). He compared the stability of rocks, cubes, Tetrapods and Accropode as shown in Fig. 2.16. He derived some important conclusions from the graph as,

- Start of damage is same for both rock and cubes. Initially stability of Accropode is higher than all other units.
- Failure of the slope was first reached by rock, then cubes, Tetrapodes and finally Accropode.

Van der Meer (1998a) also gave a stability equation for cubes as

$$\frac{H_s}{\Delta D_n} = \left(6.7 N_{od}^{0.4} / N^{0.3} + 1.0\right) s_z^{-0.1} \quad \text{... (2.50)}$$

Where, N_{od} is relative damage number, N is number of waves and s_z is the wave steepness.

Van der Meer also made a comparison study between different armor units for two levels of damage condition: no damage and severe damage. It can be observed from the graph (Fig. 2.19) that the start of damage for rock and cube armor unit is almost same and is higher for Tetrapods. Accropode have the highest $H_s/\Delta D_n$ values for start of damage compared to other armor units. The failure of slope is first reached by rock followed by cube, then Tetrapods and Accropode.

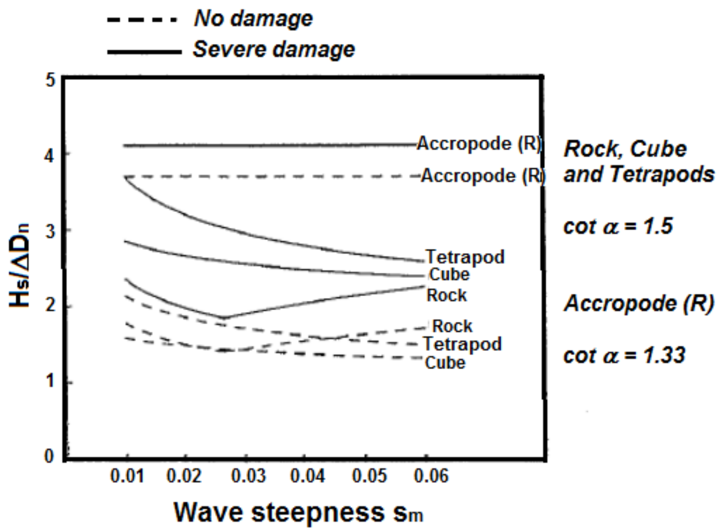


Fig. 2.19 Comparison of stability of Rocks, Cubes, Tetrapods and Accropode (Van der Meer 1998a)

Van der Meer and Heydra (1991) during their analysis found that movement of units due to rocking is concentrated around SWL. They also observed that the highest peak accelerations for Tetrapods, but the highest impact velocities for Cubes.
Based on NRA report (1991), Pilarczyk and Zeidler (1996) have given a list of requirements the armor unit has to satisfy to use it in the field.

- The armor units must provide adequate stability to withstand the severe storm waves to which the structure will be subjected.
- The unit shape should be as simple as possible to allow for easy casting, storage, handling and placement.
- The characteristic strength must be adequate to cope with the loadings and stresses induced both during construction and in service.
- The units must be sufficiently robust and durable to ensure that maintenance is negligible, this being a difficult and expensive operation for an offshore structure.
- Hydraulically the armor layer must be suitably porous, allowing a reasonable level of wave transmission.
- The structure must be economically attractive with costs kept as low as possible.

Van der Meer (1998) selected cubes as armor units for his study since these elements are bulky units which have good resistance against impact forces. The production of moulds for cubes is easier and probably cheaper than for the complicated unit shapes of Accropode and Core-loc (Van der Meer 1998). The abrasion of artificial armor unit affects the stability only in severe abrasion conditions (Allen 1998).

Trmal (2003) in his study has defined breakage of corners and edges of blocks as minor damage and can be controlled by strength of mineral and grain fabric within the material of the block.

CEM (2006) has presented a qualitative difference between interlocking and non-interlocking units in the form of a graph. It shows the influence of slope angle on stabilizing forces like gravitational force, interlocking and surface friction (Fig. 2.20).

Many researchers have stated that use of artificial armor units for berm type coastal defense structures might be feasible and economical like in the regions of Eastern Black Sea where climate is rough (Fişkin 2004).

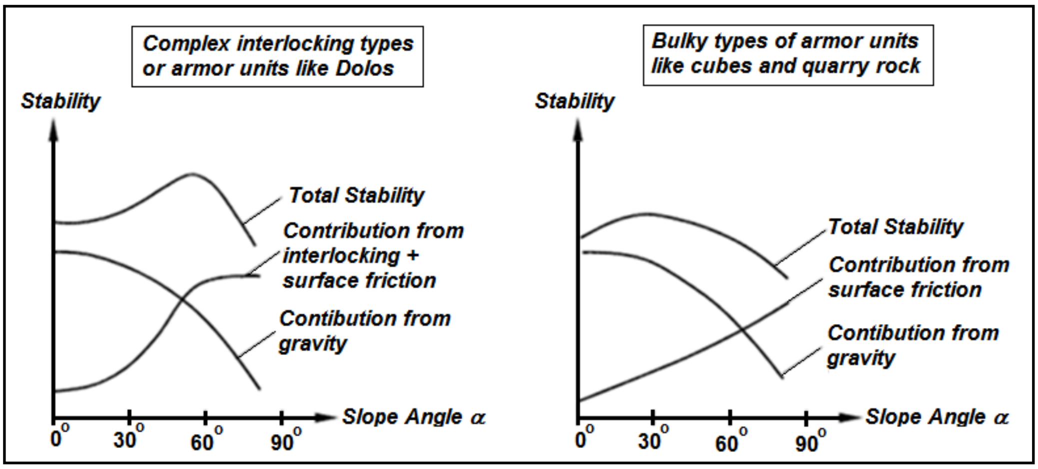


Fig. 2.20 Qualitative influence of slope angle on stability of armor units (CEM 2006)

Medina et al. (2011) compared the various aspects of cube and cubipod armor units. They found that when the equivalent drop height is lower than a certain threshold limit ($h_e < h_{e0}$), the prototypes only showed very low RLM (Relative Loss of Mass) and local edge damage. The threshold limit is 0.5m for cubes and 1.9m for cubipods. h_e is the equivalent drop height and h_{e0} is the equivalent drop height threshold limit.

Medina and Gómez-Martín (2012) suggested considering two safety factors 'Initiation of Damage (IDa)' and 'Initiation of Destruction (IDe)'. They also suggested considering different K_D values for double and single layered armors. The reason for different values was explained by them considering massive units which show tenacious failure for double layer while a brittle failure for single layer. The design K_D values and the factor of safety suggested by them are given in Table 2.8.

Table 2.8 Design K_D and global safety factors (Medina and Gómez-Martín 2012)

					Initiation of Destruction (IDe)		**Initiation of Damage (IDa)**	
Section	**CAU**	**K_D**	**layers**	**Slope**	**SF (IDe5%)**	**SF (IDe50%)**	**SF (IDa5%)**	**SF (IDa50%)**
Trunk	Cube	6	2	1:1.5	1.05	1.35	0.67	0.86
	Cubipod 2	28	2	1:1.5	1.09	1.40	0.82	0.99
	Cubipod 1	12	1	1:1.5	1.31	1.64	1.06	1.27
	Accropode	15	1	1:1.3	1.05 to 1.40	1.26 to 1.51	0.93 to 1.24	1.15 to 1.38
	Xbloc	16	1	1:1.3	1.17	1.68	1.17	1.32
Round -head	Cube	5	2	1:1.5	1.17	1.40	0.88	1.13
	Cubipod 2	7	2	1:1.5	1.19	1.36	0.99	1.18

Medina and Gómez-Martín (2012) additionally pointed out the conclusions of Burcharth and Brejnegaard-Nielsen (1986) stating that stress level in Concrete Armor Units (CAU) increases linearly with its size for static and hydrodynamic loads and is proportional to the root of the armor size for impact loads. Therefore, CAUs never break in small-scale tests; however, there may be chances of breaking at prototype scale.

Gómez-Martín and Medina (2013) have pointed out several advantages of cubes which are massive units as: high structural strength, inexpensive molds, high production rate, easy handling with pressure clamps and efficient stacking in the block yard. But, they also suggest that when cube is considered as armor, the Heterogeneous Packing (HeP) failure must be taken into account since, the units tend to move and adjust themselves in face-to-face arrangements which alter the porosity of the layer. This change in porosity lead to higher packing density below mean water level and lower packing density above and near mean water level which are critical from stability point of view.

The heterogeneous packing of the armor units may lead to block sliding failure. Davies et al. (1994) have conducted studies on block sliding and have highlighted the significance of thickness of armor layer in reducing this failure. They have concluded that thicker the armor layer greater is the resistance to sliding failures and with increase in layer thickness the sliding failure reduces.

2.12 SUMMARY

In this chapter the studies on stability of conventional rubble mound breakwater and berm breakwater have been reviewed. The various stability formulae developed by means of physical and mathematical modeling have been presented. The different types of damage parameters used for quantifying the stability of a conventional breakwater and the variables affecting them are also reviewed. The studies on wave run-up and run-down on a conventional breakwater and berm breakwater and their estimation using different formulae developed by various researchers have also been explained.

Further, a detailed introduction of berm breakwater and its types are discussed in this chapter. The stability in terms of berm recession and profile development has also been highlighted in few sections.

Additionally, from the literature, it was observed that the wave structure interaction has not been completely explained and the stability of these structures cannot be accurately modeled mathematically. The design of rubble mound breakwater has to be therefore semi-empirical relying more on laboratory studies and field experience. This has made it difficult to arrive at a design, which is both, safe from the structural standpoint as well as economical from the construction point of view. This provides a window of opportunity to take on further inquiry in the field of design of berm breakwater which is the motivational factor to take up the present work.

CHAPTER 3

PROBLEM FORMULATION AND EXPERIMENTAL DETAILS

3.1 GENERAL

Before the commencement of any experiment the details of the setup available, conditions under which it can be used are essential to be known. The wave flume facility available in Applied Mechanics Department of National Institute of Technology Karnataka can be used to generate regular waves. The scale of the model selected was obtained after conducting series of tests which is explained in detail in the forthcoming sections. Buckingham-π theorem was used for obtaining non-dimensional parameters. Experimental setup consisted of wave generator system, profiler system and wave probes.

In this chapter, problem formulation and objectives of the present study are briefed along with the details of laboratory conditions, experimental setup, dimensional analysis, hydraulic modeling, methodology and procedure adopted for the experimental investigation.

3.2 PROBLEM FORMULATION

The review of literature in the previous chapter shed light on the various studies conducted on conventional as well as berm breakwaters. The advantages of berm breakwaters over conventional breakwater were highlighted. High stability during changing wave conditions, reduced wave run-up, overtopping and wave reflection compared to conventional breakwater was also emphasized.

The studies by Priest et al. (1964), Brunn (1985), Van der Meer (1988), Ergin et al. (1989) have showed S-shaped profiles being more stable than single slope profile. Brunn (1985) has highlighted that the size of armor block for an S-shaped breakwater can be reduced to half than that required by a conventional breakwater.

Van der Meer (1988) gave the idea for measurement of damage for a statically stable and dynamically stable berm breakwater. A model was also developed to compute the profile changes due to wave attack in a dynamically stable breakwater.

The studies by Van Gent (1993) on the movement of armor stones on the slope of a berm breakwater gave an insight on the failure mechanism of the breakwater. He observed that rolling of stones was a major factor for failure.

Lissev and Torum (1996) found from their studies that a structure with its core extended into the berm was stable and they suggested it to be a viable option since, it would be cheaper compared to a traditional trapezoidal cored berm breakwater.

The stability of a berm breakwater with various profiles was studied by Juhl and Sloth in 1998. A profile with additional layer of armor protecting the berm and lower slope of a breakwater was found to be more stable than the other profiles considered for study.

A detailed report on the berm breakwater was presented by PIANC (2003) giving the studies conducted by different researchers on the various aspects of berm breakwater structure.

The breakwater using natural stone armor can't always be realized due to non-availability of required sizes of stones in the vicinity and one may have to think about artificial armor units (Neelamani and Sunderavadivelu 2003).

Joshi et al. (2006) have suggested the wave flume tests using regular waves as a versatile tool and could be used to compare merits of feasible proposals quickly and also for visualization of complex wave patterns. However, they have also mentioned that the results using regular waves would be conservative.

Number of equations to measure berm recession has been developed for a homogenous as well as layered berm breakwater (Torum et al. 1999, Sigurdarson et al. 2007, Andersen and Burcharth 2010, Moghim et al. 2011).

The importance of water level, position of berm and slope of a breakwater was emphasized by Rao et al. (2008) in their experimental studies on non-reshaped berm breakwaters.

Torum et al. (2012) have stressed the use of economical optimization for assessing the risk levels of berm breakwaters which includes loss due to damage of structure and loss of lives due to its failure.

Recent studies by Van Gent et al. (2012) and Van Gent (2013) have suggested usage of concrete armor units in berm and lower slope of a berm breakwater.

The research on berm breakwater as explained earlier have shown that the studies were mainly concentrated considering natural rock as armor unit and the equations developed were also for the same. The equations thus developed are valid under certain ranges and cannot be used universally. Further, with no definite design principles available for the design of berm breakwaters, the design is mainly dependent on the field experience. The alternative numerical and mathematical models developed have not helped in reducing the physical model studies as emphasized by many researchers.

In order to arrive at a more realistic knowledge of seaward profiles for which wave intensity and likelihood of breakwater damage are minimal, a laboratory study of shallow water wave action on reshaped berm breakwater is proposed in the present investigation. The present study includes understanding the mechanism and performance of berm breakwater with artificial armor units and to evolve an optimal breakwater section.

3.3 OBJECTIVES OF THE PRESENT WORK

The objectives of the present research are to experimentally investigate the influence of varying wave parameters on the stability of cube armored,

1. Conventional breakwater.
2. Statically stable reshaped berm breakwater with design weight and varying berm widths.
3. Statically stable reshaped berm breakwater with reduced armor weights, varying berm widths and thickness of primary layer; and
4. Optimal design of statically stable reshaped berm breakwater.

3.4 DIMENSIONAL ANALYSIS

The present model study involves a berm breakwater structure made of concrete cube as primary armor unit. The modeling of wave-structure interaction in the present study is difficult. The major portion of wave energy will be lost over the berm portion, before it attacks the upper slope due to breaking of waves. This phenomenon of wave breaking and the downrush of waves which causes return current are difficult to express mathematically and one has to depend upon experimental investigations. The results of such investigations are more useful when expressed in dimensionless forms. To arrive at such relations of different variables, dimensional analysis is carried out. Dimensional analysis is a rational procedure for combining physical variables into dimensionless parameters, thereby reducing the number of variables that need to be considered.

3.4.1 Predominant variables

The predominant variables considered for dimensional analysis in the present investigation are listed in the Table 3.1.

Table 3.1 Predominant variables in the analysis of breakwater stability

Predominant Variable		Dimension
Wave Parameters	Wave Height (H)	L
	Water Depth (d)	L
	Wave Period (T)	T
	Deepwater Wave Length (L_o)	L
	Number of waves (N)	$M^oL^oT^o$
	Run-up (R_u)	L
	Run-down (R_d)	L
Structural parameters	Armor unit Weight (W)	M
	Nominal Diameter (D_{n50})	L
	Berm Width (B)	L
	Berm position (h_b)	L
	Cotangent of breakwater slope ($\cot\alpha$)	$M^oL^oT^o$
	Mass density of armor units (ρ_a)	ML^{-3}
	No. of primary armor layer (n)	$M^oL^oT^o$
	Permeability of the structure (P)	$M^oL^oT^o$
Fluid Parameters	Mass Density (ρ_w)	ML^{-3}
	Kinematic Viscosity (υ)	$ML^{-1}T^{-1}$
External Effects	Acceleration due to Gravity (g)	LT^{-2}

3.4.2 Details of dimensional analysis

For deep water wave conditions L and T are related by

$$L = \frac{gT^2}{2\pi} \quad \text{...} \quad (3.1)$$

The term gT^2 is used to represent the wave length L during dimensional analysis. This is because if L is used it would be depth specific while gT^2 is independent of depth and represents the deep water wave characteristics which can easily be transformed to shallow waters depending upon local bathymetry.

The stability of rubble mound breakwaters can be described by static stability or dynamic stability. The static stability of the breakwater is defined by the term damage level (S) or damage number (N_{od}). The dynamic stability is defined by the profile parameters as length and angles. The stability of berm breakwaters are mainly characterized based on the erosion in the berm portion which is termed as berm recession (R_{ec}).

Considering the damage level (S) of the rubble mound breakwater and berm recession (R_{ec}) of berm breakwater, which is dependent on several parameters, their relationships can be expressed as,

$$S = f\,(W, H, T, d, g, \rho_a, \rho_w, D_{n50}, R_u, R_d, \cot\alpha, P) \quad \text{..................} \quad (3.2)$$

$$R_{ec} = f\,(W, H, T, d, B, h_b, g, \rho_a, \rho_w, D_{n50}, R_u, R_d, \cot\alpha, P) \quad \text{..............} \quad (3.3)$$

By the application of Buckingham's π theorem, an equation of the form shown below is obtained.

$$S = f\{\, H/\Delta D_{n50}, H_o/gT^2, R_u/H_o, R_d/H_o, d/gT^2, \cot\alpha) \quad \text{...........................} \quad (3.4)$$

$$R_{ec}/D_{n50} = f\{\, H/\Delta D_{n50}, H_o/gT^2, h_b/d, B/d, R_u/H_o, R_d/H_o, d/gT^2, \cot\alpha) \quad \text{....} \quad (3.5)$$

Where,

S	Damage level
R_{ec}/D_{n50}	Dimensionless berm recession
R_{ec}/B	Relative berm recession
$H/\Delta D_{n50}$	Hudson's stability number, N_s
H_o/gT^2	Deepwater wave steepness, s
B/d	Relative berm width
h_b/d	Relative berm position
$\tan\alpha/(s)^{-0.5}$	Surf similarity parameter, ξ
d/gT^2	Relative water depth
R_u/H_o	Relative run-up

R_d/H_o	Relative run-down
$\cot\alpha$	Cotangent of breakwater slope

3.5 SIMILITUDE CRITERIA AND MODEL SCALE SELECTION

The basis of all physical modeling is similitude of the model and prototype. Similitude is said to be achieved when all factors that influence the phenomenon are in proportion between the prototype and the model. According to Hughes (1993), model similitude can be achieved by: i) Calibration ii) Differential equation iii) Dimensional analysis iv) Scale series

The method of calibration is appropriate for movable bed models. The method of differential equations is more applicable when the differential equations considered are reasonably accurate. Scale series is mostly used to establish the scaling criteria for a complex phenomenon, and one has to be extremely careful in analyzing the results from the model tests and extrapolating them to prototype.

In the present study, the similitude is achieved by the method of dimensional analysis. The similitude is achieved between the prototype and the model with the help of non-dimensional parameters of the phenomenon. These non-dimensional parameters must be of the same range for both model and the prototype. Considering the wave climate off Mangalore coast, in the present study similitude is achieved by considering the non-dimensional parameter, wave steepness H/gT^2 as given in Table 3.2. In the laboratory using the existing facilities of the two-dimensional wave flume, regular waves of heights ranging from 0.03 m to 0.24 m and periods ranging from 1 s to 3 s can be produced.

Table 3.2 Wave parameters of prototype and model

Wave Parameters	H (m)	T (sec)	H/gT^2
Prototype	1 to 5.4	8 to 12	0.00070 to 0.0086
Model	0.030 to 0.24	1.0 to 3.0	0.00033 to 0.0244

For selecting a model-scale the range of wave heights and wave periods that can be generated in the wave flume accounting the wave climate off Mangalore coast are considered. The appropriate length scale range for rubble mound stability tests is between 1:5 and 1:70 (Hudson et al. 1979). To simulate the field conditions of wave height, period and nominal diameter of armor stone, by the application of Froude's law, a geometrically similar scale of 1:30 was selected (Table 3.3). This scale is within the scale selected for rubble mound breakwater model tests conducted in majority of the laboratories around the world and is good enough to give reasonable and satisfactory results compared to those of the prototype (Le Mehaute 1976, Palmer and Walker 1976, Jensen and Klinting 1983, Stive 1985 and Hughes 1993).

Many researchers had observed that there will be scale effect on armor stability for lower Reynolds number. For many years, it was considered that the Reynolds number of flow through armor layer must be greater than $3x10^4$ for no modeling errors to arise. But, Owen and Briggs (1986) from their model studies conducted with different scales suggested that, the Reynolds number can be as low as $8x10^3$ or even $3x10^3$ before any significant errors can arise. In the present work, effect of Reynolds number is not considered, because, flow in the primary armor layer is considered to be turbulent as $R_e > 3x10^3$.

Table 3.3 Selection of model scale

Scale	H (m)		T (sec)		D (m)
	1	5.4	8	12	1
1:10	0.1	0.54	2.53	3.8	0.1
1:20	0.050	0.27	1.79	2.68	0.05
1:30	**0.033**	**0.18**	**1.46**	**2.19**	**0.03**
1:40	0.025	0.135	1.26	1.90	0.025

3.6 DESIGN CONDITIONS

The wave climates off the Mangalore coast as given by KREC Study Team (1994) are considered while planning the present experimental investigation. During the monsoon, the maximum recorded wave height off Mangalore coast is about 4.5 m to 5.4 m. During fair weather season wave height hardly exceeds 1 m. Predominant wave period is 8 s to 11 s. Occasionally, during the fair weather season, wave periods up to 15 s are observed. Hence, for the design of conventional rubble mound breakwater model an equivalent of prototype design wave of height 3 m is assumed, while a maximum wave of height up to 4.8 m and period of 8 s to 14 s are considered for model study.

3.7 EXPERIMENTAL SETUP

3.7.1 Wave flume

The physical model study for regular waves was conducted in a two dimensional wave flume available in Marine Structures laboratory of Applied Mechanics Department, National Institute of Technology Karnataka, Surathkal, India. The wave flume is 50 m long, 0.71 m wide and 1.1 m deep. It has a 41.5 m long smooth concrete bed. About 15 m length of the flume is provided with glass panels on one side. It has a 6.3 m long, 1.5 m wide and 1.4 m deep chamber at one end where the bottom hinged flap (or wave paddle) generates waves.

A ramp is provided between flume bed level and generating chamber for generation of waves. A series of vertical asbestos cement sheets are provided as wave filter spaced at about 0.1 m centre to centre parallel to length of the flume. The flap is controlled by an induction motor of 11 KW power at 1450 rpm. This motor is regulated by an inverter drive (0 – 50 Hz) rotating in a speed range of 0–155 rpm.

Waves of height ranging from 0.08 m to 0.24 m heights and periods from 0.8 sec to 4.0 sec in a maximum water depth of 0.5 m can be generated with this facility. Fig. 3.1 shows the line diagram of wave flume.

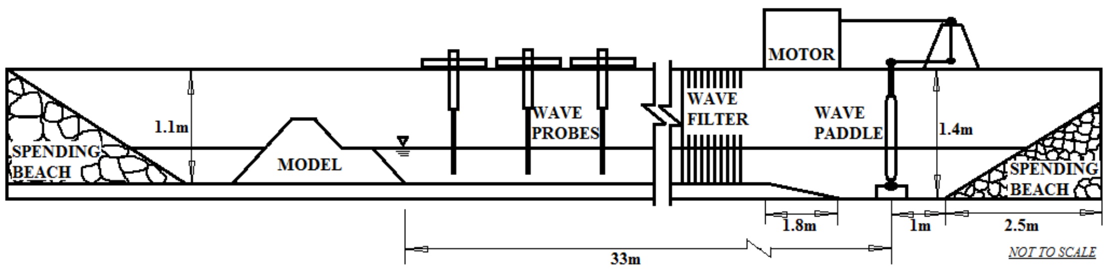


Fig. 3.1 Details of wave flume facility

3.7.2 Wave probes

The capacitance type wave probes are used in the present study. The accuracy of the measurements is 0.001 m. The probes were used to record the incident wave characteristics. The spacing of probes and decomposition of incident and reflected waves from superposed waves recorded by wave probes was done using the three probe method suggested by Isaacson (1991).

3.7.3 Surface profiler system

The surface profiler system consists of a wooden frame with nine sounding probes at 0.075 m centre to centre. This is mounted on the railings on side walls of the flume and can be moved along to take the profile of the breakwater model.

3.7.4 Calibration of test facilities

Calibration of the experimental set up and instruments were undertaken frequently to check and ensure accuracy. The method of calibration of each component is given below.

3.7.4.1 Wave flume

The aim of calibration of wave flume is to evaluate a relationship between frequency of the inverter and wave period and eccentricity and wave height for a particular water depth. The regular waves of height (H) ranging from 0.10 m to 0.16 m with varying periods (T) from 1.6 to 2.6 s for different water depths were required for the experiment. Desired wave period can be generated by changing of frequency through inverter drive. Wave height for a particular wave period can be produced by changing the eccentricity of bar chain on the fly wheel. Combinations that produced secondary waves in the flume are not considered for the experiments. Fig. 3.2 shows a typical calibration chart for water depth of 0.35 m.

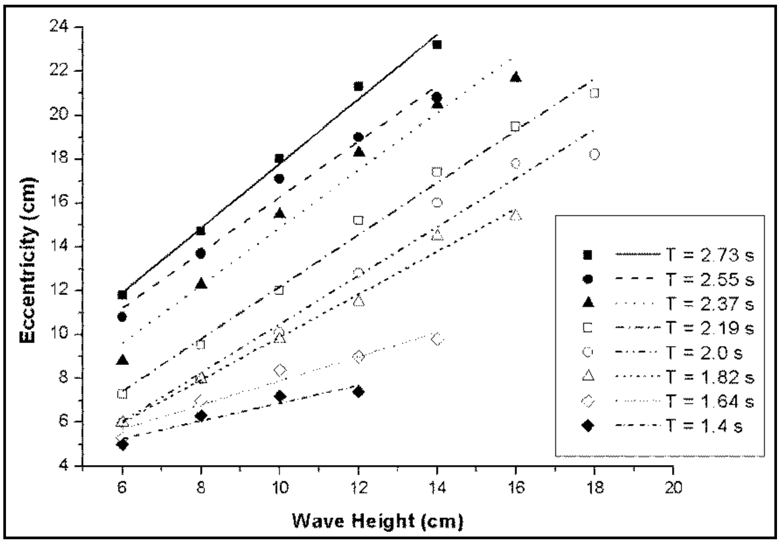


Fig. 3.2 Calibration chart for a water depth of 0.35 m

3.7.4.2 Wave probes

The probes were originally calibrated by the manufacturer. However, the output is expected to show minor variations depending on the salinity and temperature of the water used in the flume. Hence, the probes were subjected to static immersion tests and the relationship between the water level and the output voltage was determined and recorded. The variation of voltage with water level is shown in Fig. 3.3. The calibration was undertaken daily before and after the experiments. The free surface of water is considered as zero mark in x-axis and the probe will be moved from the bottom tip to the top tip inside the water to record the change in voltage.

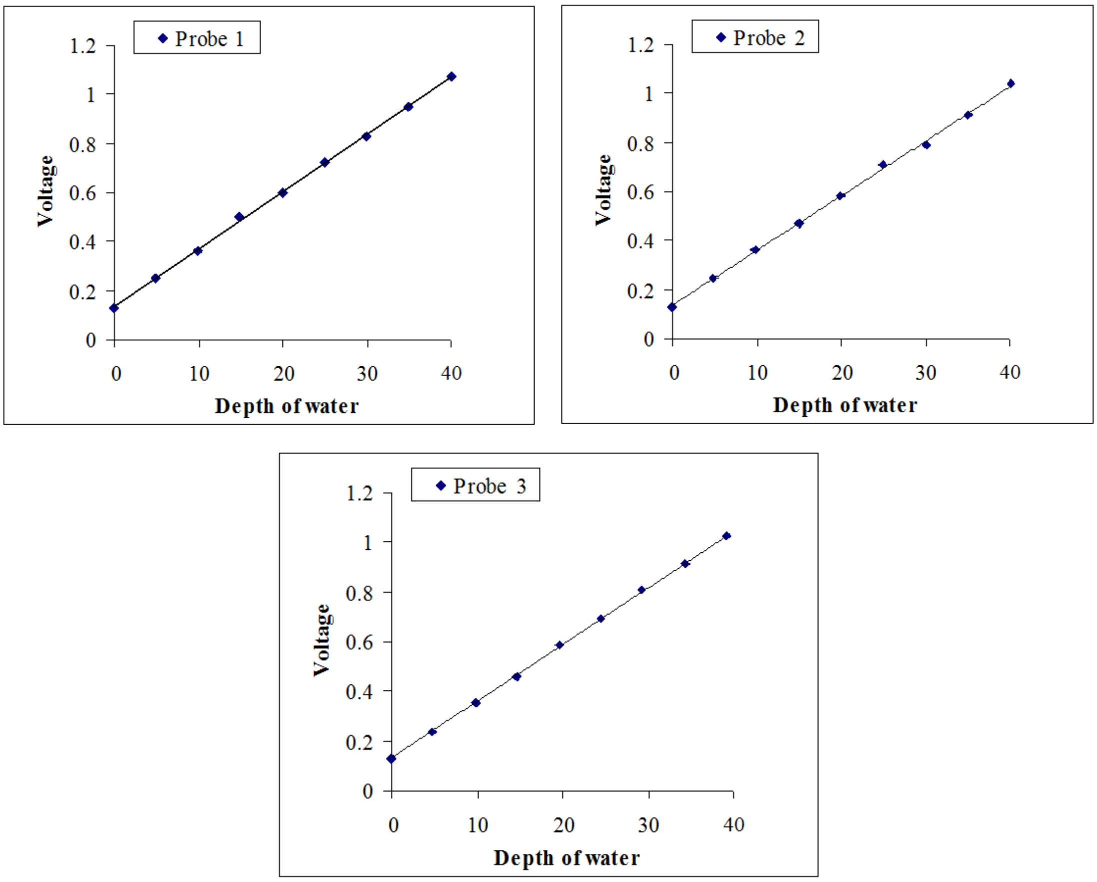


Fig. 3.3 Calibration of wave probes

3.8 BREAKWATER MODEL SECTIONS

3.8.1 Conventional breakwater

The conventional breakwater model of trapezoidal section was built with a uniform slope of 1:1.5. The breakwater was constructed with concrete cubes as primary armor.

The K_D for the concrete cubes was derived as 5.5 from the following relation (National River Authority, 1991, as quoted by Pilarczyk and Zeidler, 1996)

$$\frac{K_D \text{ of Tetrapods}}{K_D \text{ of Cubes}} = 1.5 \quad \text{.......................................} \quad (3.6)$$

The unit weight (W_{50}) of 106 g for primary armor cube was determined using Hudson's formula (Eq. 3.7) for a design wave height (H) of 0.10 m.

$$W = \frac{1}{K_D \cot \alpha} \frac{H^3 \gamma_r}{\Delta^3} \quad \text{...} \quad (3.7)$$

The armor cube size (D_{n50}) was determined using Eq. 3.8 and for the present case it was found to be 0.0353 m.

$$D_{n50} = \left(\frac{W}{\gamma_r} \right)^{1/3} \quad \text{..} \quad (3.8)$$

The weight of the stones in the secondary layer was taken as 10.6 g (W/10). A minimum crest width of three armor units was maintained which was 0.10 m and crest height was 0.70 m to avoid wave over topping. The core was designed with quarry dust of 300 microns size.

The thickness of the primary and secondary layer was determined by the equation

$$t = nK_\Delta \left(\frac{W}{\gamma_r} \right)^{1/3} \quad \text{..} \quad (3.9)$$

where, t = thickness of armor layers

n = number of layers of the armor units

K_Δ = layer coefficient

γ_r = specific weight of armor material (concrete cube=2.4 g/cc)

The average layer thickness of 0.118 m was provided for primary layer and an average thickness of 0.055 m was provided for secondary layer. Fig. 3.4 shows a typical cross section of conventional rubble mound breakwater. The model characteristics are tabulated in Table 3.4.

Table 3.4 Conventional breakwater model characteristics

Variable	Expression	Value
Slope	α	1V: 1.5H
Armor type	--	Concrete cube
Mass Density	γ_r	2.4 g/cc
Stability coefficient	K_D	5.5
Relative mass density of armor unit	Δ	1.4
Nominal size of armor cube	D_{n50}	0.0353 m
Armor cube weight	W_{50}	106 g
Layer coefficient	K_Δ	1.1
Crest width	--	0.10 m
Crest height	--	0.70 m
Thickness of primary armor layer	--	0.118 m
Thickness of secondary armor layer	--	0.055 m
Porosity of primary armor	P_1	47%
Porosity of secondary armor	P_2	39%
Porosity of core	P_3	36%

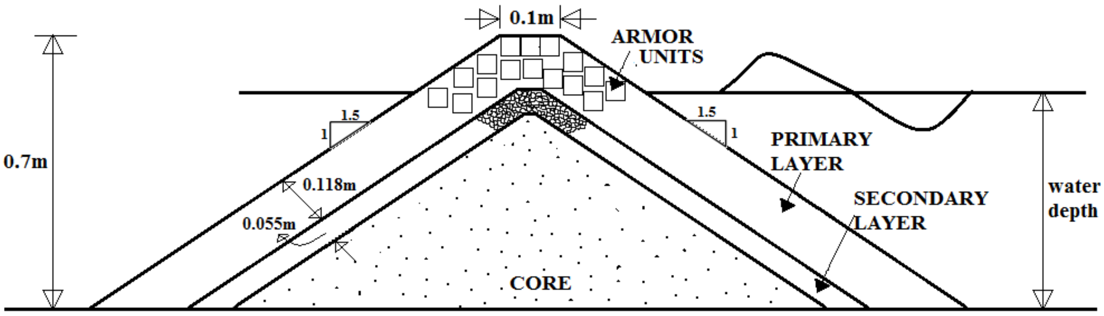


Fig. 3.4 Details of test model of conventional breakwater

3.8.2 Berm breakwater

Berm breakwater model was also constructed with a slope of 1V: 1.5H with cubes as armor unit. Various models were constructed with different armor weights. The armor weights considered were 1.0W, 0.75W, 0.6W and 0.5W, where W represents the weight of armor obtained using Hudson formula (i.e., 106 g) for a design wave height of 0.10 m. The armor cube size and thicknesses of primary and secondary armor layer were calculated using Eq. 3.8 and Eq. 3.9, for the reduced armor weights considered. The crest height and crest width were provided same as conventional breakwater of 0.70 m and 0.10 m respectively. A range of berm widths varying from 0.30 m to 0.45 m was considered in the present study. The berm was maintained at a constant height of 0.45 m from sea bed. Fig. 3.5 shows the section of berm breakwater. The berm breakwater characteristics are tabulated in Table 3.5.

Table 3.5 Berm breakwater model characteristics

Variable	Expression	Value
Slope	α	1V: 1.5H
Armor type	--	Concrete cube
Mass Density	γ_r	2.4 g/cc
Armor cube weight	W_{50}	106, 79.5, 63.6, 53 g
Size of armor cube	D_{n50}	0.0353, 0.0325, 0.0298, 0.0285 m
Berm width	B	0.30, 0.35, 0.40, 0.45 m
Berm position from sea bed	h_b	0.45 m
Crest width	--	0.10 m
Crest height	--	0.70 m

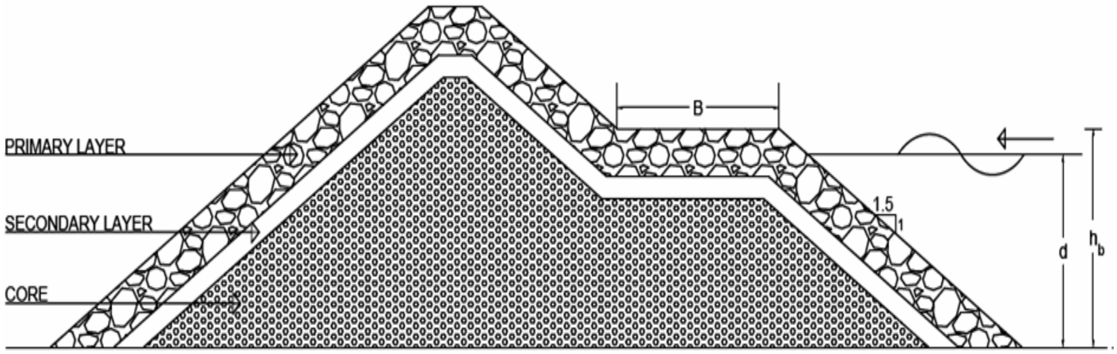


Fig. 3.5 Details of test model of berm breakwater

3.8.3 Model construction

The breakwater model was constructed on the flat bed of wave flume at a distance of 32 m from the generator flap. The cross section of breakwater denoting layers was drawn on the glass panel. The core material was placed and formed to the required level and after well compacting; the secondary layer and the primary layer were constructed to the marked level. The placement of primary armor units was random over the secondary layer. The primary armor layers were colored red, white and grey respectively to reduce the surface roughness (Hughes 1993). Pipes are provided below the breakwater test section to balance the water levels on seaside and leeside (Diskin et al. 1970). A same slope of 1:1.5 was maintained on both sides of the breakwater.

3.8.4 Wave characteristics

The breakwater models were tested for varying wave characteristics (test conditions) as given in Table 3.6. The models were subjected to normal wave attack of height ranging from 0.10 m to 0.16 m and of periods varying from 1.6 s to 2.6 s in a depth of water (d) of 0.30 m, 0.35 m, 0.40 m and 0.45 m. Storm duration of 3000 waves (which is equivalent to an actual storm of 6.67 hours to 10.83 hours) was considered since more than 80% to 90% of the total damage would have already inflicted by that time and equilibrium would have established (Van der Meer and Pilarczyk 1984 and Hegde 1996).

Table 3.6 Wave characteristics

Variable	Expression	Value
Wave height	H	0.10, 0.12, 0.14, 0.16 m
Wave period	T	1.6, 2.0, 2.6 s
Water depth	d	0.30, 0.35, 0.40, 0.45 m
Storm duration	N	3000 waves
Angle of wave attack	-	90^0
Wave type	-	Regular

3.9 CASTING OF CONCRETE CUBES AND ITS PROPERTIES

Various tests like standard consistency, initial and final setting time, fineness of cement and specific gravity of cement were conducted on cement. Similarly, sand was also tested for its specific gravity and fineness modulus. Compressive strength of 28 days cured cubes was determined. The obtained results are compared with the standard Indian Standard (IS) code specifications and all of them are within the limits specified in the code. Table 3.7 shows the different tests performed on cement and sand, the obtained values and standard limits.

Table 3.7 Results of tests on cement and sand

Test	Value obtained	IS code value	IS code
Cement			
Standard consistency	30%	28% – 32%	IS: 8112 - 1989
Initial setting time	66 min	Not less than 30 min	
Final setting time	330 min	Not more than 600 min	
Fineness of cement	300 m^2/Kg	> 225 m^2/Kg	
Specific gravity	3.1	3.1 – 3.15	
Sand			
Specific gravity	2.64	Nearer to 2.65	IS: 2720 (Part III/Sec 1) - 1980
Fineness modulus	3.29	2.3 – 3.5	IS: 2386 (Part I) – 1963
Cube			
Compressive strength of 28 days cured cube	37.5 N/mm^2	> 35 N/mm^2 (for M35 grade)	IS: 456 - 2000

Concrete cubes were casted using 1:3 cement mortar with a water cement ratio of 0.4. To maintain the density of concrete as 2.4 g/cc, a part of sand was replaced by iron ore filings. The Table 3.8 gives the different sand: iron ore filings ratios and its mass densities. A mass density of 2.396 g/cc was obtained for a ratio of 80: 20 which was considered for the present study.

Table 3.8 Sand: Iron ore ratio in 1:3 cement mortar

Trial No.	Sand: Iron Ore	Mass density of cube in g/cc
1	90:10	2.05
2	87:13	2.178
3	85:15	2.288
4	83:17	2.378
5	**80:20**	**2.396**
6	78:22	2.342

The cubes were casted with the above parameters and cured for 28 days before constructing the model. The porosity of the armor layers was determined as per IS 2386 (Part III) - 1963 and the porosity of primary armor, secondary layer and core were 47%, 39% and 36% respectively.

3.10 METHODOLOGY

The wave flume is filled up with fresh water to the desired level and calibrated to produce the selected wave height and period without putting the model. The measuring instruments are also calibrated. The model is constructed at 32 m away from the generator flap.

Initially, 1:30 scale model of conventional single breakwater of 1V:1.5H sloped trapezoidal cross section is constructed on the flume bed with primary concrete cube armor for a non-breaking design wave of 0.10 m. This model is tested for stability with regular waves of heights 0.10 m to 0.16 m and periods 1.6 s to 2.6 s in water depths of 0.30 m, 0.35 m and 0.40 m. The conventional breakwater characteristics are shown in Table 3.4. In the second phase berm breakwater models are constructed with cube armor with a slope of 1V:1.5H (above and below the berm). The models are tested for varying structural and sea state parameters as shown in Table 3.5 and Table 3.6 respectively. All the models are tested for stability along with the wave run-up and run-down.

After the tests, finally, the optimum berm breakwater configuration is deduced. The procedure is depicted in the flow chart shown in Fig. 3.6.

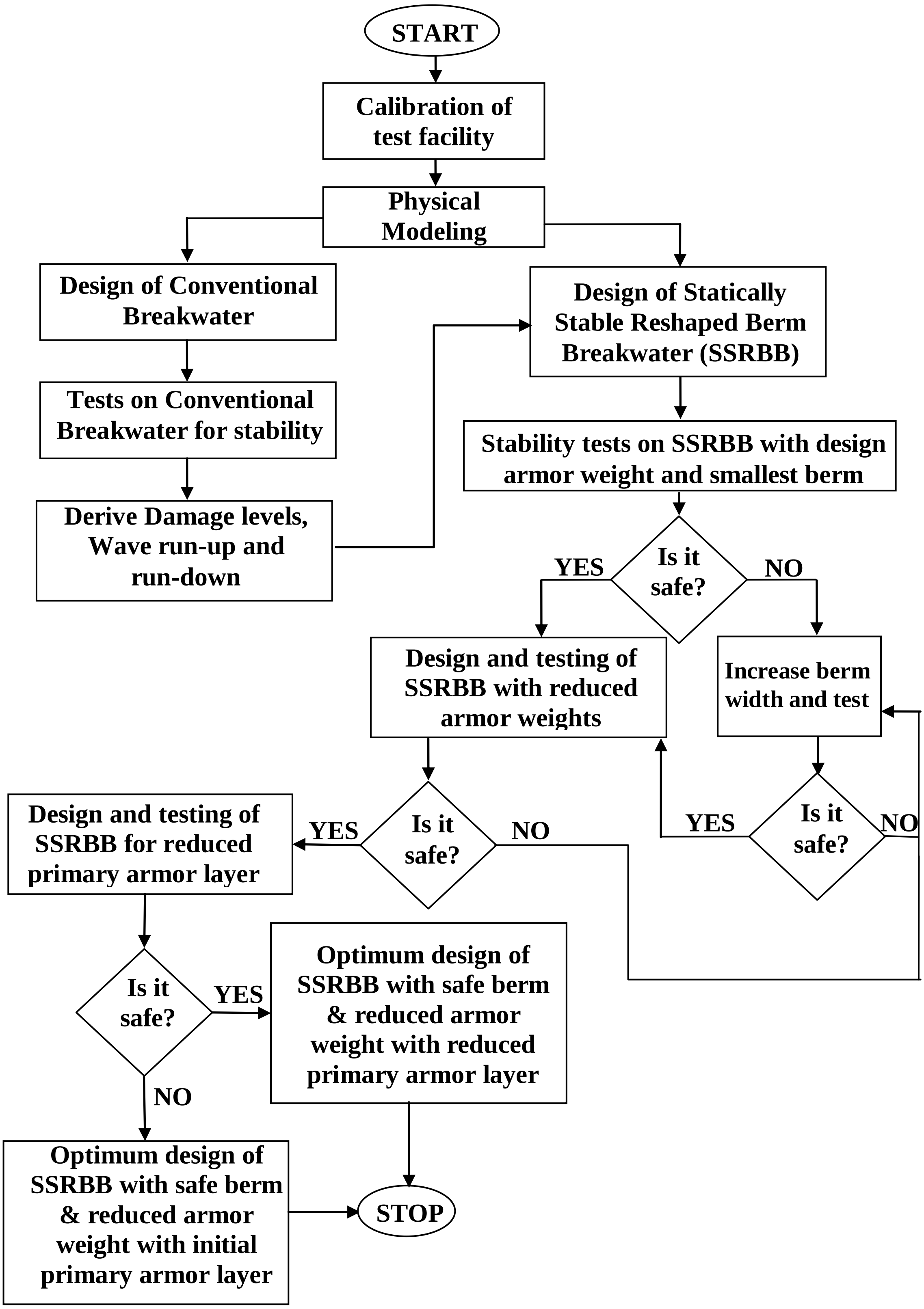


Fig. 3.6 Flowchart of the methodology

3.11 TEST CONDITIONS

The present experimental investigations are carried out with the following test conditions:

1. The sea bed is rigid and horizontal and it is assumed that the sediment movement does not interfere with the wave motion and do not affect the model performance.
2. The waves of each burst are periodic and monochromatic.
3. Waves are generated in short burst of five waves.
4. Between wave bursts there are brief intervals to allow wave energy to dampen out.
5. Secondary waves generated during the test are not considered.
6. The density difference between freshwater and seawater is not considered.

In the lab, the reflected waves from the breakwater strike the wave paddle and are almost totally re-reflected and create waves which may not represent the prototype condition truly. Further, the re-reflected waves may add-up or reduce the incident wave height and period thus resulting in undesired wave heights and periods. This results in erroneous output which is not desirable. This problem is eliminated by conducting experiments with series of wave bursts, such that, each burst of waves ending before re-reflected waves can again reach the testing section of the wave flume (Hughes 1993).

3.12 TEST PROCEDURE

To minimize uncertainty the experiment has to be properly planned, experimental procedures and extrapolation methods should be standardized. Sources of errors and magnitude of errors have to be minimized.

3.12.1 Sources of errors and precautions to minimize error

The following sources are identified which may cause error in the experimental study.

1. Error in linear dimensions: The model is constructed with an accuracy of linear dimensions up to ±1.0 mm, which may contribute errors in between 0.2% to 0.3%.
2. Error in wave height measurement: The least count of the wave probe is 1.0 mm and may contribute to an error of 2% to 6% in the incident wave height.

3. Error due to change in water level: The water level is checked at the beginning and end of every day's work and maintained within ±2 mm of the required level. This may contribute to an error of 0.6% to 1.0%.

These errors are expected to be compensative in nature and hence, there is no significant effect if they are neglected. However, the following precautions are taken for minimizing the errors:

1. The model is constructed, as per the standard procedure, with a largest possible model with a scale of 1:30.
2. The depth of water in the flume is maintained exactly at the required level and was continuously monitored. Average variation of 2 mm was found after a full day of model testing. Any drop in the water level of more than 2 mm was immediately corrected.
3. Before the commencement of the experiments, calibration of flume and wave probes without the placement of model were undertaken to determine the proper wave height to assign to a particular combination of generator stroke and wave period. The wave heights used in the test runs are obtained during calibration. This will exclude the losses due to interference of flume bed, sidewalls and reflection and therefore, eliminates these error sources.
4. Waves were run in short bursts of five during the tests. Between wave bursts there will be brief interval to allow reflected wave energy to dampen out.
5. All the wave characteristics were measured with more than 35 readings and were statistically analyzed. Similar exercise was repeated for other parameters such as wave run-up and run-down over the breakwater.
6. Tests were started by surveying newly constructed slope with profiler, which becomes reference survey for comparison of subsequent surveys.

3.12.2 Procedure for experimental study

After carefully studying the model test conditions and procedures adopted by various investigators, the following procedure is adopted for the present study.

1. All the test models were constructed, in the flume, at a distance of 32 m from the wave generator flap.
2. The wave flume was filled with tap water and the entire experimental setup including the wave probes were calibrated for each depth of water.

3. The breakwater models were constructed and their initial profiles were taken for comparison of subsequent surveys (final profile).
4. The models were tested for varying wave characteristics as listed in Table 3.6. Initially the models were tested for smaller wave height of 0.1 m for different wave periods of 1.6, 2.0 and 2.6 s. The wave heights were then increased by 0.02 m till they reach 0.16 m for all the wave periods. 3000 waves or the exposure of the secondary layer, whichever occurred earlier, were run for each trial.
5. During the initial study of each model wave run-up and run-down were measured. After each trial the final profile was taken to calculate the damage expressed in terms of berm recession (R_{ec}).

3.13 MEASUREMENTS

3.13.1 Measurement of wave heights

Three capacitance type wave probes are used during the experimental work to measure the wave heights. The probes are kept on the seaside of model for acquiring incident wave heights (H_i). The spacing between probes was kept near to one third of the wave length. During the experimentation, the signals from wave probes are verified online and recorded by the computer through the data acquisition system. The statistics namely, the ranges of standard deviation and coefficient of variation for all wave heights recorded for all depths of water in the entire model tests of the present study are listed in Table 3.9.

Table 3.9 Statistics of all wave heights recorded

Depth of water d (m)	Range of statistical parameters of wave heights	
	Standard deviation σ (m)	Coefficient of variation c
0.45	0.08 – 0.26	0.006 – 0.065
0.40	0.07 – 0.10	0.005 – 0.011
0.35	0.10 – 0.21	0.011 – 0.042
0.30	0.06 – 0.14	0.004 – 0.019

3.13.2 Measurement of wave run-up and run-down

Wave run-up was measured as the vertical distance above the SWL of the maximum wave up rush when a wave impinges on the breakwater model. The vertical distance below the SWL up to the minimum elevation attained by wave on the breakwater slope was taken as the wave run-down. The wave run-up and run-down are measured over the breakwater on the graduated scale fixed over the glass panels of the flume. The statistics of all wave run-up and run-down recorded for the entire model tests of the present study are listed in Table 3.10 and Table 3.11 respectively.

Table 3.10 Statistics of all wave run-ups recorded

Depth of water d (m)	Range of statistical parameters of wave run-ups	
	Standard deviation σ (m)	Coefficient of variation c
0.45	0.03 – 0.09	0.0008 – 0.0071
0.40	0.03 – 0.09	0.0006 – 0.0064
0.35	0.03 – 0.11	0.0007 – 0.0072
0.30	0.02 – 0.10	0.0005 – 0.0066

Table 3.11 Statistics of all wave run-downs recorded

Depth of water d (m)	Range of statistical parameters of wave run-downs	
	Standard deviation σ (m)	Coefficient of variation c
0.45	0.06 – 0.11	0.0033 – 0.0085
0.40	0.05 – 0.11	0.0027 – 0.0087
0.35	0.05 – 0.10	0.0028 – 0.0101
0.30	0.05 – 0.10	0.0021 – 0.0079

3.13.3 Measurement of breakwater damage

The recession of the berm and the exposure of secondary layer on the sea side slope or the berm reflect the damage of a structure. If R_{ec}/B is less than one then structure is safe otherwise unsafe. Further, the structure was considered damaged with the exposure of secondary layer even if $R_{ec}/B < 1$. Surface profiler system was used for obtaining profile before and after the test of the breakwater. Intermediate profiles were also obtained to study the effect of storm duration.

3.14 UNCERTAINTY ANALYSIS

Generally, whenever experimentation is involved, there is a possibility of some errors creeping in while making measurements. Some of the errors involved in the present study are already explained in previous sections. Further, it is also observed that the experimental results as well as extrapolated values for prototype obtained from different test facilities vary considerably. In such events, it becomes necessary for a researcher to provide the upper and lower margins of the calculated results with a fair amount of confidence level. Such a study of providing a confident margin is called as uncertainty analysis. Uncertainty analysis is the process of calculating uncertainty of a value that has been calculated from several measured quantities. It describes the degree of goodness of a measurement or experimentally determined result. With the help of uncertainty analysis it is possible to conduct experiments in a scientific manner and predict the accuracy of the result (Misra 2001). The details of the analysis are illustrated in Appendix I.

3.15 PHOTOS OF EXPERIMENTAL SETUP AND MODELS

Plate 3.1 A view of wave flume with berm breakwater model

Plate 3.2 Mould for cube casting

Plate 3.3 Berm breakwater model

Plate 3.4 Conventional breakwater model

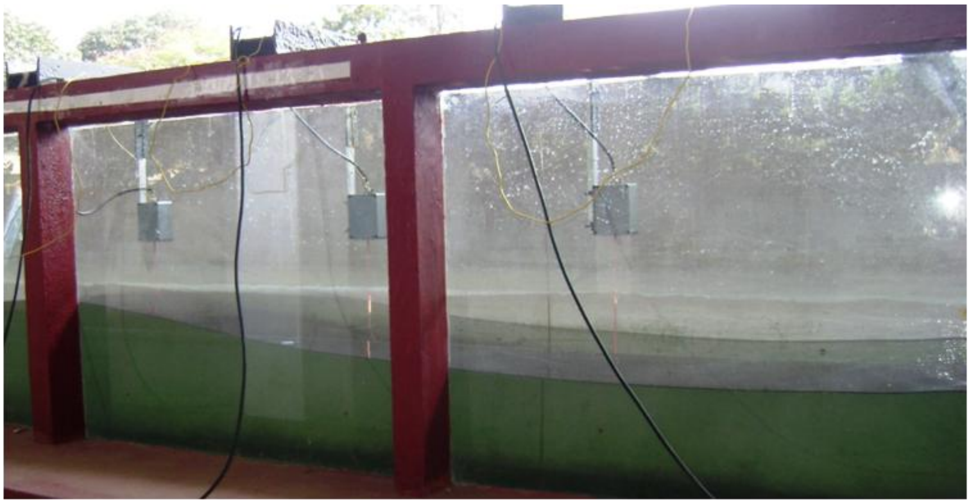


Plate 3.5 Wave probes for data acquisition

Plate 3.6 Wave Structure Interaction

Plate 3.7 Surface profiler system

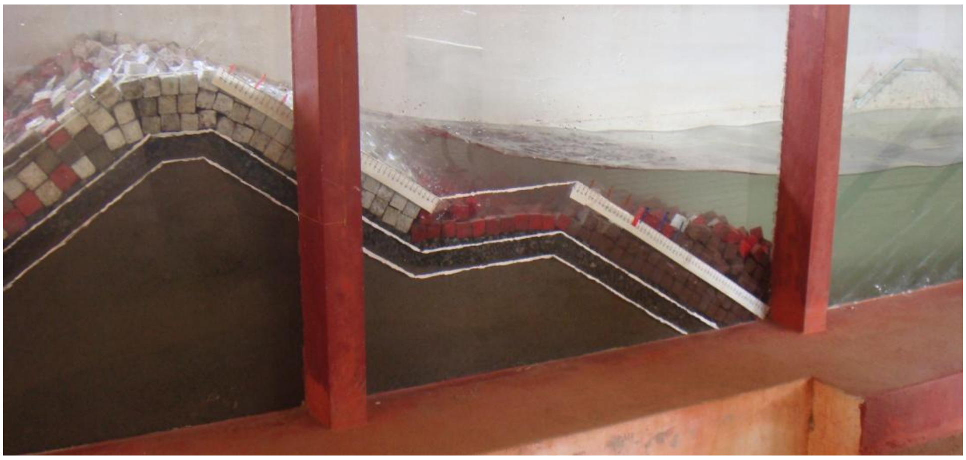


Plate 3.8 Wave up rush along the structure

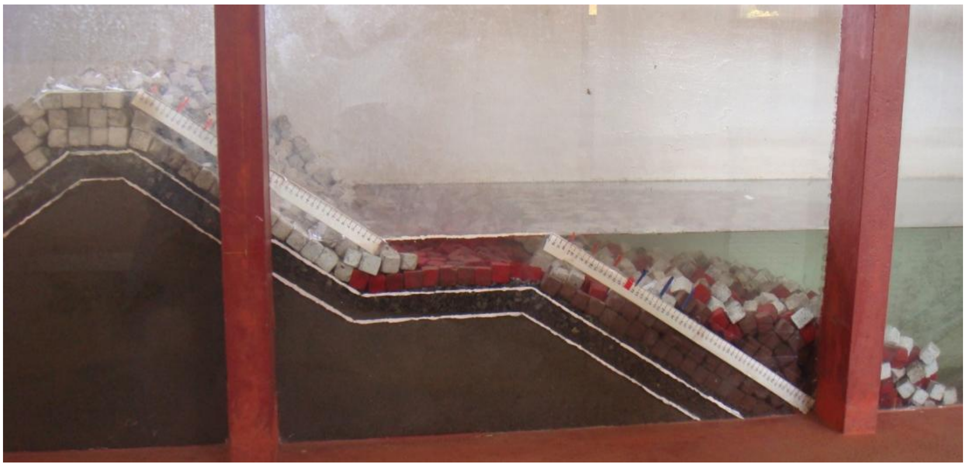


Plate 3.9 Reshaped profile of berm breakwater after storm duration

CHAPTER 4

RESULTS AND DISCUSSION

4.1 GENERAL

After the completion of experiments, the results obtained have to be interpreted accurately to know the performance of a structure. In this chapter the effect of various sea state parameters as well as structural parameters on the stability, wave run-up and run-down of the berm breakwater are analyzed in detail.

A safe and economical structure has to stand and be stable for all the wave conditions that it is being tested with a minimum cross sectional area. The stability analysis is carried out to know the vigor of a structure to withstand the onslaught of waves. Wave run-up studies help in knowing the highest water level a wave reaches and dislocation of primary armor while wave run-down studies aid us in relating the movement of armor and hence the stability of the primary layer of the breakwater. Understanding the effect of various sea and structural parameters is essential in knowing the behavior of the structure and design an optimized structure which is safe as well as economical for the wave conditions considered.

4.2 SUMMARY OF MODEL STUDY

The starting point of the experimental studies was the trapezoidal cross section model made of cube armor constructed with a scale of 1:30 on the flat bed of the flume. A uniform slope of 1V:1.5H was maintained with primary armor weight of 106 g (i.e., nominal diameter, D_{n50}, of 0.0353 m) for a non-breaking design wave of 0.10 m. Its crest width was 0.1 m and height 0.70 m. The secondary armor of mean weight of about 10.6 g and core of quarry dust of size about 300 microns was designed. The porosities of the primary armor, secondary armor and core are 43%, 39% and 36% respectively. The primary armor units were painted with different colors and placed in bands of heights of 0.2 m to 0.3 m to track their movement during damage. In the first phase this conventional breakwater model was tested. The breakwater model characteristics are listed in Table 3.4.

The model was constructed at a distance of 32 m from the generator flap. The cross section of breakwater denoting layers was drawn on the glass panel. Pipes were placed over the flume bed, at the breakwater test section, to balance the water levels on seaside and leeside. The core material was placed and formed to the required level by light compacting with hands. Then, the secondary and primary layers were constructed to the marked level. The technique adopted in the present work was keyed and fitted placement. The primary armor layers were painted to reduce the surface roughness. In the rubble mound breakwater model, the armor units placed around SWL were painted in white and in the zone above it was painted with red color while in the zone below the units were painted with grey color to identify the damage zone after the test. Further, the details of the breakwater model construction are explained in Chapter 3.

Before the model tests were started, the experimental set up along with the wave probes was calibrated to find the required wave heights which, were assigned to a particular combination of generator stroke and wave period, for depths of water of 0.30 m, 0.35 m, 0.40 m and 0.45 m. The model was subjected to regular waves of height (H) varying from 0.1 m to 0.16 m of a range of period (T) varying from 1.6 s to 2.6 s generated in water depths (d) of 0.30 m to 0.45 m. The model was tested for varying wave characteristics as shown in Table 3.6. The model was subjected to normal attack of regular waves. The waves were generated in bursts of five waves with brief interludes to allow reflected wave energy to dampen out. During the test the number of incident waves, their heights and periods, wave run-up and run-down over the breakwater slope and armor unit movements were recorded. Further details of test procedure are explained in Chapter 3.

In the second phase different berm breakwater models constructed with cube armor with a slope of 1V:1.5H (above and below the berm) was tested. The experimental works proceeded with the changes in armor unit sizes and berm widths to see their effects on the stability of the structure. In each experiment, 12 individual test waves forming a storm having the significant wave height (0.1 m to 0.16 m) and period (1.6 s to 2.6 s) were generated. The armors in the primary layer were painted similarly as in the case of conventional breakwater, with red color for units in the upper slope above the berm, grey color for units in the lower slope below the berm and white for the units in the berm. The berm in berm breakwaters were mostly constructed above still water level. The breakwater model characteristics are listed in Table 3.5. Initial and final profiles of the breakwater structure before and after the tests were also taken additionally to the other readings.

Initially, the conventional breakwater was tested for stability and the results obtained was used to design the statically stable reshaped berm breakwater with design armor weight, minimum primary layer thickness of 2 and a smallest berm width of 0.30 m. The model was tested for stability and was found that the structure was safe for a higher berm width of 0.35 m. Hence, much higher berm widths of 0.40 m and 0.45m were not tested for this armor weight. Next, the armor weight was reduced by 25% of the design weight and the stability was analyzed with berm widths varying from 0.30 m to 0.45 m and 3 layers of primary armor. The models with 0.30 m and 0.35 m berm widths were not safe but the structure was safe for higher berm widths of 0.40 m and 0.45 m. Further, for the same reduced armor weight, the thickness of primary armor was reduced to 2 and the stability was tested. Model with berm width 0.45 m was the only safe structure obtained during these tests. Hence, further studies were conducted with 3 layers of primary armor and 0.45 m berm width for more reduced armor weight. The models with 40% and 50% reduced armor weight were not safe for all the test conditions. So, the stability was tested with increased primary layer thickness of 4. Even then the models were not safe with 40% and 50% reduced armor weights.

During these tests some typical observations were made. It was observed that with the decrease in armor weight and berm width the berm recession, wave run-up and run-down was found increasing. Also, the increase in water depth and wave height increased the recession of berm, wave run-up and run-down. Further, it was observed that shorter period waves caused higher berm recession, lower wave run-up and run-down. In the next few sections details of the results are discussed for the some of the critical test conditions.

4.3 MECHANISM OF BREAKWATERS

4.3.1 Conventional Breakwater

In conventional breakwater, the waves impinging on the structure will up-rush along the slope while some energy of the waves will be lost between the porous layer of the primary layer. The up-rushing waves will lift the unstable armor units while the down-rush of the same wave will bring down the armor layer.

4.3.2 Berm Breakwater

In case of berm breakwater which has a horizontal berm, the interaction of waves with the structure is completely different compared to a conventional breakwater and the interaction has to be analyzed for various wave and structural parameters.

(a) Different berm elevation: For a small water depth below the berm, the waves impinge on the lower slope and the effect of the berm is not felt at all. This becomes a case similar to conventional breakwater. Further, for a water level nearer or equal to the berm level, the waves impinge on the berm and dissipate most of its energy in the berm portion itself without causing much up-rush and down-rush and the berm itself may get damaged without damaging the breakwater slope. When water level is above the berm, the wave height determines the damage to the breakwater. For a steeper wave, it will break near the berm which may not cause much damage to the upper slope. A less steep wave will directly impinge on the upper slope, but, the energy within the wave would be less thus inflicting less damage to the breakwater.

(b) Different berm width: The variation in berm width also affects the damage inflicted onto the structure. For longer berm widths, the berm will sacrifice itself by absorbing the wave energy and suffering damage, but, will safeguard the upper slope where as for a shorter berm width, the berm will be sacrificed and the upper slope will also be damaged which may cause the failure of entire structure.

(c) Thickness of primary layer: The variation of thickness of primary layer may also affect the damage to the structure. As the number of layers increases, more energy is dissipated within the layers thus reducing damage to the structure.

4.4 STUDIES ON CONVENTIONAL BREAKWATER

4.4.1 Effect of deepwater wave steepness (H_o/gT^2) on damage level (S)

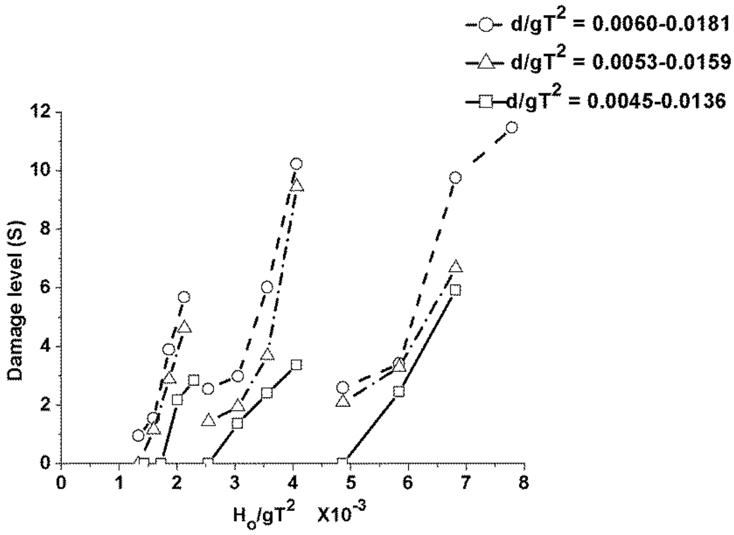


Fig. 4.1 Influence of deepwater wave steepness (H_o/gT^2) on Damage level (S) for different water depths

Fig. 4.1 shows the influence of deep water wave steepness (H_o/gT^2) on damage level (S) for different water depths. It is clear from the graph that S increases with the increase in H_o/gT^2 and d/gT^2. The highest S of 11.5 is observed for a water depth of 0.40 m (d/gT^2 = 0.00181) and a wave height of 0.16 m for a wave period of 1.6 s (H_o/gT^2 = 0.00778). It is also observed that for a 0.30 m (d/gT^2 = 0.0045 - 0.0136) depth of water and a wave height of 0.10 m, S is zero for all the wave periods (H_o/gT^2 = 0.00143, 0.00254, 0.00486). The high damage for steeper waves is due to the dislodging of armor unit from its position by the waves and subsequent pulling down of them even before they settle. The damage level for lower water depths ranged from ‘no damage’ to ‘intermediate damage’ while for higher water depths a complete failure was observed for many of the test conditions.

4.4.2 Effect of deepwater wave steepness (H_o/gT^2) on relative wave run-up (R_u/H_o)

The influence of deep water wave steepness parameter (H_o/gT^2), on relative run-up (R_u/H_o) for increasing ranges of depth parameter (d/gT^2) i.e., varying wave climate in depths of water of 0.3 m, 0.35 m and 0.4 m, is shown in Fig. 4.2.

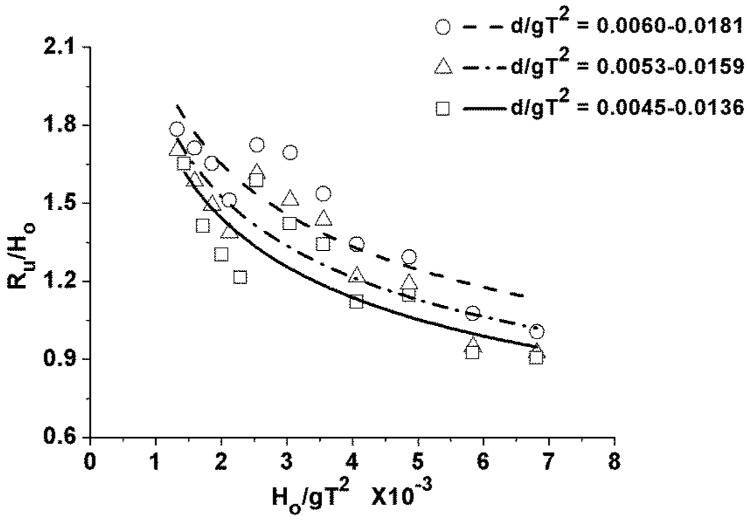


Fig. 4.2 Influence of H_o/gT^2 on R_u/H_o for different water depths

The relative run-up decreases with an increase in wave steepness and there is no significant impact of d/gT^2. For $1.1x10^{-3} < H_o/gT^2 < 6.7x10^{-3}$ and all ranges of d/gT^2, R_u/H_o vary from 0.905 to 1.785. In the present study, it is observed that, run-up is relatively higher for $1.1x10^{-3} < H_o/gT^2 < 4x10^{-3}$, when, $4 < \xi < 4.25$. This is because, in this range of ξ, surging breakers occur. As water depth decreases from 0.40 m to 0.35m and 0.30 m, the wave run-up decreases by 4.6% to 7.9% and 7.5% to 9.9% respectively.

4.4.3 Effect of deepwater wave steepness (H_o/gT^2) on relative wave run-down (R_d/H_o)

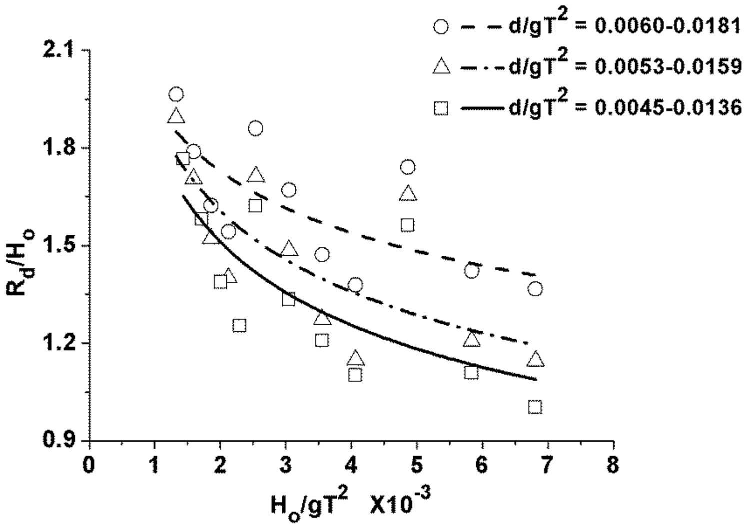


Fig. 4.3 Influence of H_o/gT^2 on R_d/H_o for different water depths

The influence of deep water wave steepness parameter (H_o/gT^2), on relative run-down (R_d/H_o) for increasing ranges of depth parameter (d/gT^2) i.e., varying wave climate in depths of water of 0.3 m, 0.35 m and 0.4 m, is shown in Fig. 4.3. The relative run-down decreases with an increase in wave steepness and decreasing d/gT^2 similar to relative wave run-up. For $1.1x10^{-3} < H_o/gT^2 < 6.7x10^{-3}$ and all ranges of d/gT^2, R_d/H_o vary from 1.003 to 1.965. As water depth decreases from 0.40 m to 0.35m and 0.30 m, the wave run-down decreases by 3.8% to 19.4% and 11.2% to 36.3% respectively.

4.5 STUDIES ON BERM BREAKWATER

4.5.1 Effect of reduction in armor weight on berm recession, wave run-up and run-down

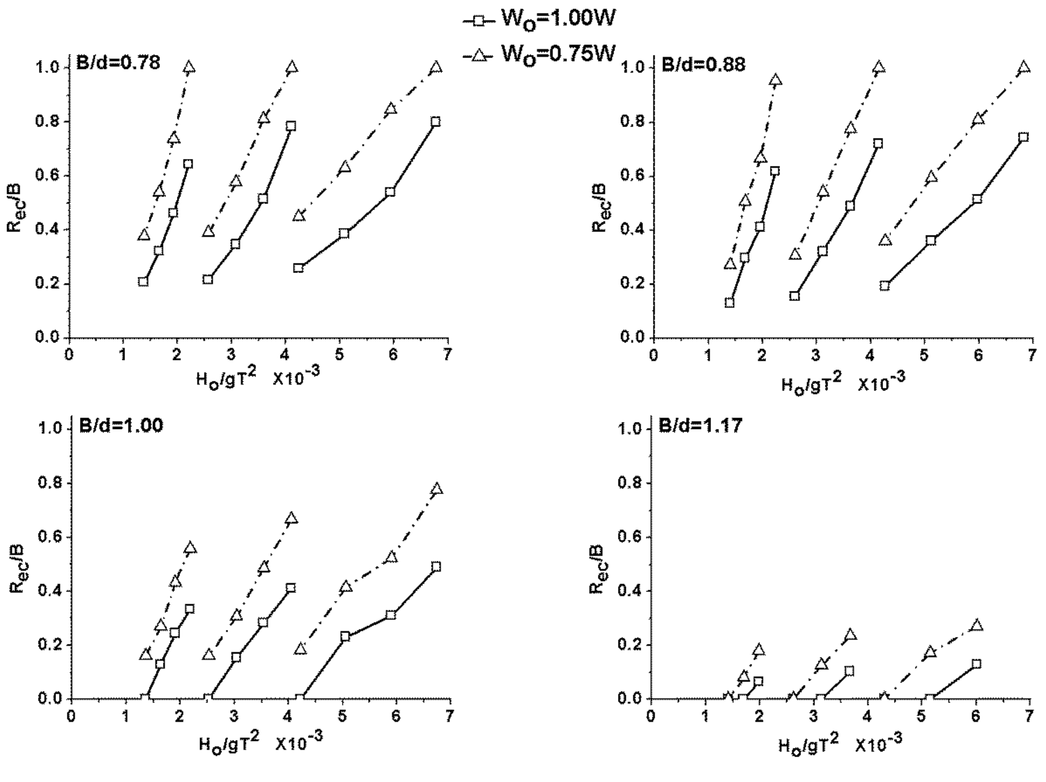


Fig. 4.4 Influence of H_o/gT^2 on R_{ec}/B for a 0.35 m berm width and different water depths

Fig. 4.4 shows the comparison of berm recession for design (W_o = 1.0W) armor weight and reduced (W_o = 0.75W) armor weight models for a berm width of 0.35 m. It is observed from the figure that, as the armor weight decreases there is an increase in relative berm recession (R_{ec}/B) with increasing H_o/gT^2 for all conditions considered. The design armor weight considered for the model was safe for all the conditions, but when the armor weight was reduced by 25%, for water depths 0.45 m (B/d = 0.78) and 0.40 m (B/d = 0.88) the breakwater was not safe for the wave height of 0.16 m for all wave periods. The damage was in the form of complete erosion of berm with no secondary layer exposure in these models. Shorter wave periods caused more recession compared to longer wave periods as observed in the Fig. 4.4. As armor weight is reduced from W to 0.75W, the berm recession increased by 20% to 100% for all the conditions considered.

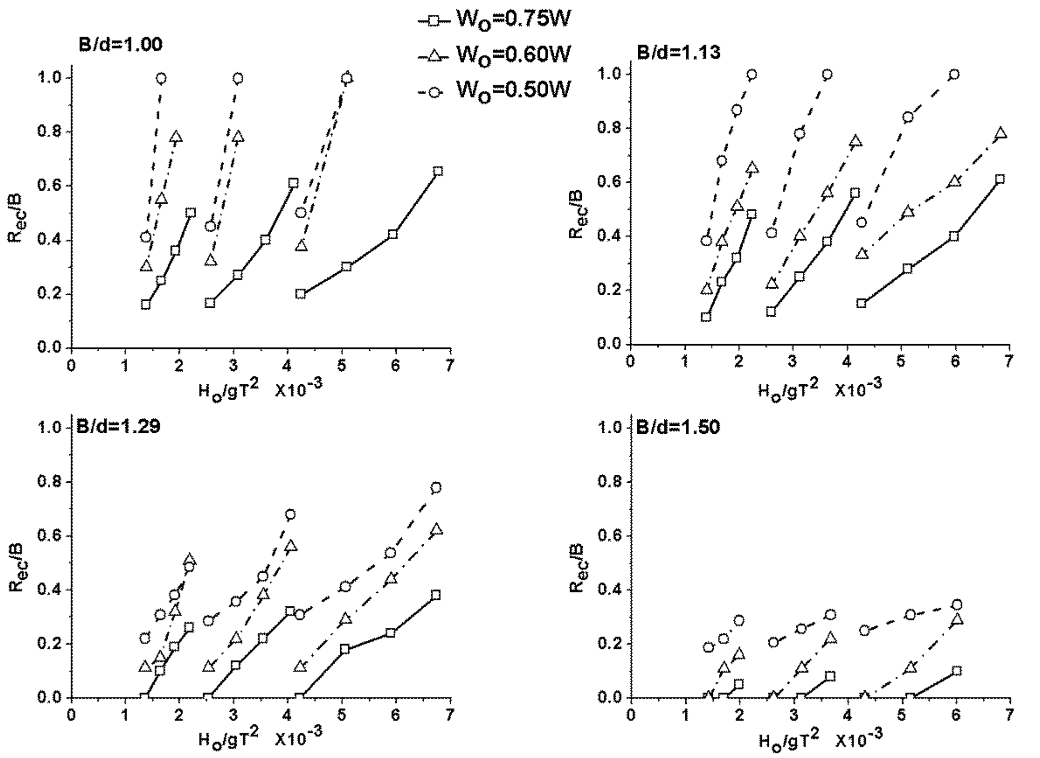


Fig. 4.5 Influence of H_o/gT^2 on R_{ec}/B for different water depths and 0.45 m berm width

Fig. 4.5 shows the typical variation of relative berm recession (R_{ec}/B) with deepwater wave steepness (H_o/gT^2) for different reduced armor weight models in varying depths of water (d) for a constant berm width (B) of 0.45. The reduced armor weights considered are 0.25W, 0.6W and 0.5W. Since, from Fig. 4.4 it is observed that design armor weight models were safe for a berm width of 0.35 m, higher berm width of 0.45 m were not tested with these models. Hence, design armor weight models are considered for comparison in Fig. 4.5. In Fig. 4.5 also same trend of increasing R_{ec}/B with decreasing armor weight and B/d ratio can be observed. It is also observed from figure that for water depths of 0.45 m (B/d = 1.0) and 0.40 m (B/d = 1.13) the R_{ec}/B was found to be greater than one in 0.6W and 0.5W armor weight models for many of the wave conditions. In these models both types of damage to the structure have been observed. Secondary layer was exposed in 0.6W and 0.5W armor weight models for 0.14 m and 0.16 m high wave and almost all the wave periods. The secondary layer was exposed within short storm duration of 1500 waves. The complete erosion of berm was observed in 0.5W armor weight models for a wave height of 0.12 m at all the wave periods and 0.6W armor weight model for a 0.12 m wave height at a period of 1.6 s.

The 0.75W armor weight models were completely safe for all the wave conditions with the berm recession less than berm width ($R_{ec}/B < 1$) with no exposure of secondary layer. As the armor weight is reduced from 0.75W to 0.6W and 0.5W, the R_{ec}/B increases by 26% to 100% and 47% to 100% respectively.

The failure of the structure for higher reduction in armor weight is due to smaller size of the unit which reduces the porosity in the armor layer. Since, cube is a bulky armor, the wave attack is resisted by its weight and with the reduction in weight, as in the present case, the resistance also decreases thus inducing higher damage to the structure. Also, with the reduction in size, the armor units get closely packed thus decreasing the porosity between the units and subsequently reducing the wave energy dissipation which increases the damage on the structure. Further, the increased recession for waves of shorter period is due to the fact that the initial waves floats the armor unit from its position and before it settles back the remaining waves dislodge them from their position thus inflicting damage to the structure. Along with this, the decrease in water depth decreases the berm recession since, the waves carry less energy in lower depths which is efficiently dissipated by the berm.

The influence of deepwater wave steepness (H_o/gT^2) on relative run-up (R_u/H_o) and relative run-down (R_d/H_o) for different reduced armor weights (W_o) and relative berm width (B/d) are shown in Figs. 4.6 and 4.7 respectively. These tests were conducted with a constant berm width of 0.45 m. It is clear from the figures that both R_u/H_o and R_d/H_o decrease with the increase in H_o/gT^2, while, there is no significant impact of B/d and reduction of armor weight.

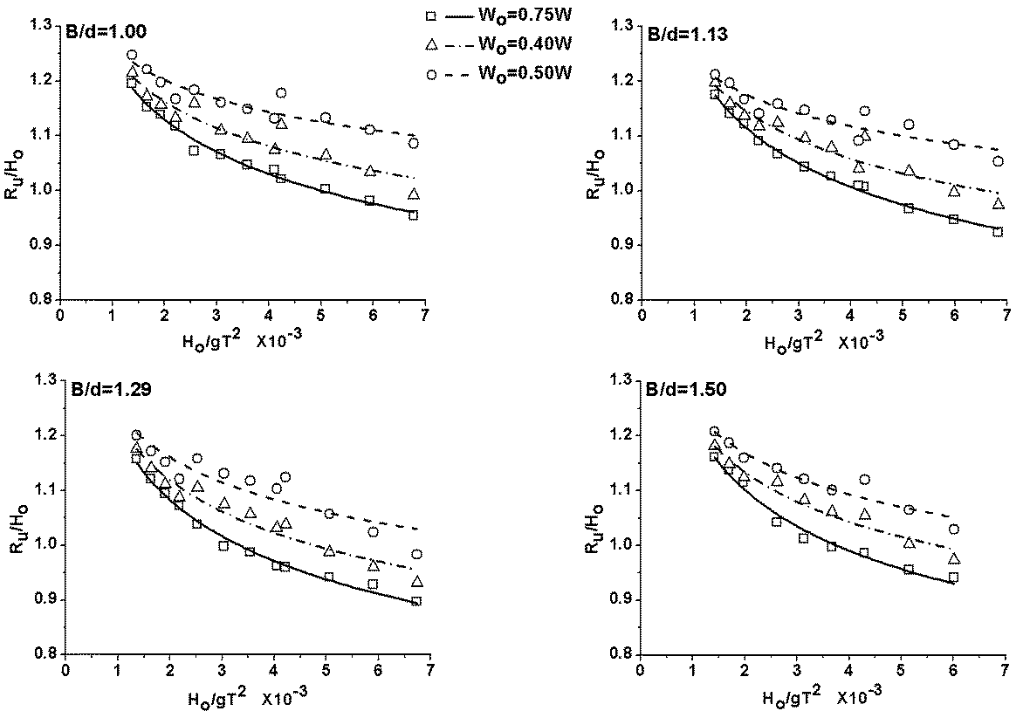


Fig. 4.6 Influence of H_o/gT^2 on R_u/H_o for different water depths and 0.45 m berm width

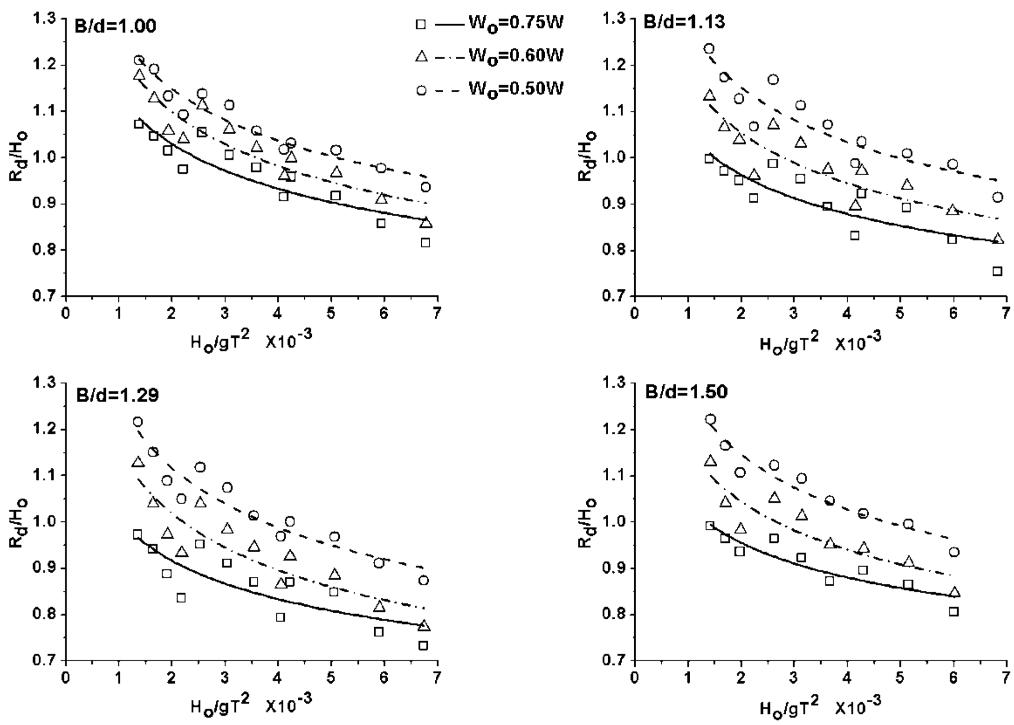


Fig. 4.7 Influence of H_o/gT^2 on R_d/H_o for different water depths and 0.45 m berm width

From the Fig. 4.6, it is observed that berm absorbs more energy for less steeper waves and hence the wave run-up converges irrespective of armor weights and water depths. The reduction in wave run-up and run-down for higher wave steepness is due to the action of berm as ‘stilling basin’ which controls the inflow and outflow of water through the structure which reduces the run-up and run-down (Bruun and Johannesson 1976). The wave run-up and run-down increased with higher reduction in armor weight because of decreased porosity in the armor layer. As the water depth decreases, both wave run-up and run-down decreases due to the lower energy of the wave for lesser depths of water.

For all wave steepness and B/d, as armor weight reduces from 0.75W to 0.6W and 0.5W, the relative wave run-up increases by 1.7% to 3.8% and 4.4% to 9.6% respectively. Similarly, the wave run-down also increases by 5.3% to 9% and 13.3% to 16.3% respectively with the reduction in armor weight from 0.75W to 0.6W and 0.5W. From the discussions it is observed the reduction in armor weight does have a significant effect on berm recession while wave run-up and wave-down are not affected.

4.5.2 Effect of varying berm widths on berm recession, wave run-up and run-down

The influence of deepwater wave steepness (H_o/gT^2) on relative berm recession (R_{ec}/B) for 0.75W armor weight models in different water depths can be observed in Figs. 4.8 (a) – (d). 25% reduction in armor weight with different berm widths is considered for the comparison, since, this was the only model tested for all the berm widths. R_{ec}/B was found to be increasing as the H_o/gT^2 increased which was also observed in section 4.5.1. From the Fig 4.8 it is noticed that larger the width of berm lower is the berm recession. It is also observed that in all the cases, as berm width increases from 0.40 m to 0.45 m, the reduction in berm recession is less, showing that further increase in berm width will not be effective.

Fig. 4.8 (a) shows the influence of H_o/gT^2 on R_{ec}/B for a water depth of 0.45 m. For B/d = 0.67 and 0.78 the failure is observed at wave heights of 0.16 m and 0.14 m for almost all the wave periods. The models with berm widths of 0.45 m (B/d = 1.0) and 0.40 m (B/d = 0.89) are found safe with R_{ec}/B being less than 1.0 for all the test conditions. The berm recession is least for 0.45 m berm width in comparison with other berm width models for all the wave steepness considered. In a 0.45 m depth of water as B/d increased from 0.67 to 0.78, 0.89 and 1.0, the R_{ec}/B decreased by 0 to 18%, 26.8% to 44.5% and 34.6% to 53.7% respectively.

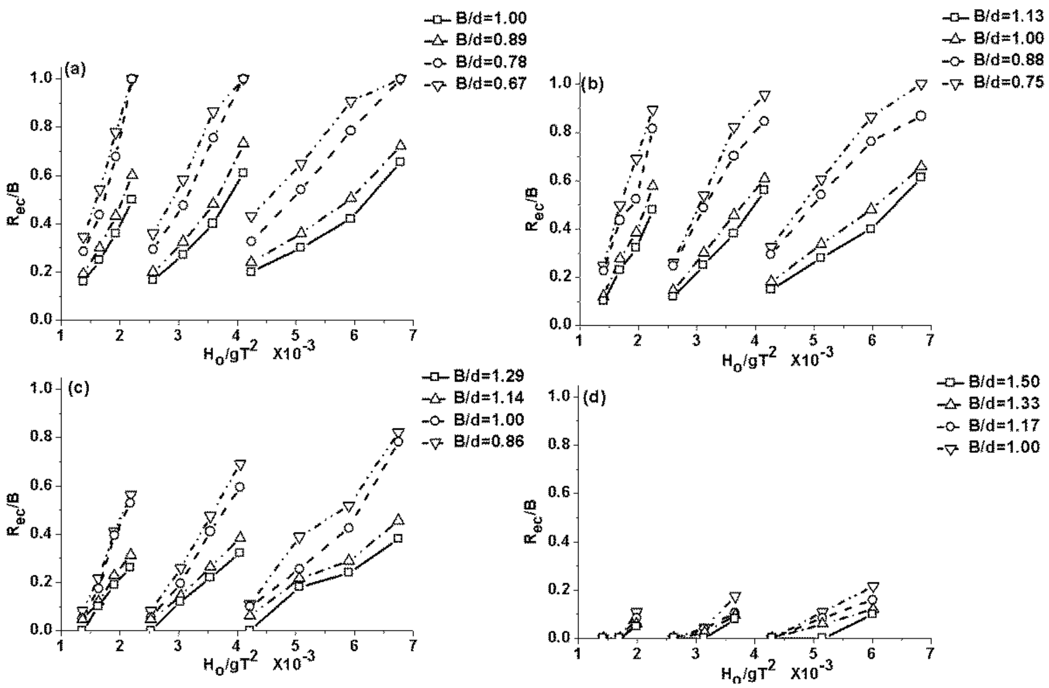


Fig. 4.8 (a) – (d) Influence of H_o/gT^2 on R_{ec}/B for 25% reduction in armor weight and different water depths

In Fig. 4.8 (b), for water depth of 0.40 m, the model with B/d = 0.75 (B = 0.30 m) for a wave 0.16 m and 1.6 s (H_o/gT^2 = 0.0067) showed failure with R_{ec}/B = 1.0. All the other models were found safe for the considered wave conditions. In 0.40 m water depth, the R_{ec}/B decreased by 8.1 to 13.3%, 34.3% to 51.2% and 38.8% to 59.3% respectively as B/d increased from 0.75 to 0.88, 1.0 and 1.13.

All the models in Fig. 4.8 (c) were found safe for the entire wave conditions considered in 0.35 m deep water. A maximum R_{ec}/B of 0.8 was observed for 0.30 m berm width (B/d = 0.86) model for wave of 0.16 m and 1.6 s (H_o/gT^2 = 0.0067). In a 0.35 m depth of water as B/d increased from 0.86, 1.0, 1.14 and 1.29, the R_{ec}/B decreased from 12.6% to 39.7%, 36.7% to 44.6% and 53.7% to 100% respectively.

Fig. 4.8 (d) also shows that R_{ec}/B is less than 1.0 for the wave conditions in a water depth 0.30 m. Wave height of 0.16 m could not be generated since they were breaking before reaching the structure.

Considering all the water depths, as the berm width increased from 0.30 m to 0.35 m and 0.35 m to 0.40 m, there was a maximum reduction of berm recession upto 16.3% and 80% respectively, while, with the increase in berm width from 0.40 m to 0.45 m, a maximum reduction in berm recession of 18.5% is observed, which shows that further increase in berm width would not help significantly in reducing the erosion of berm. The failure and high berm recession for smaller berm width in all the cases is due to the reduced area available for dissipation of wave energy in the berm portion. Also, the reduced berm width and increased wave height caused the waves to directly attack the upper slope and dislodge the armor units there by inflicting high damage to the structure.

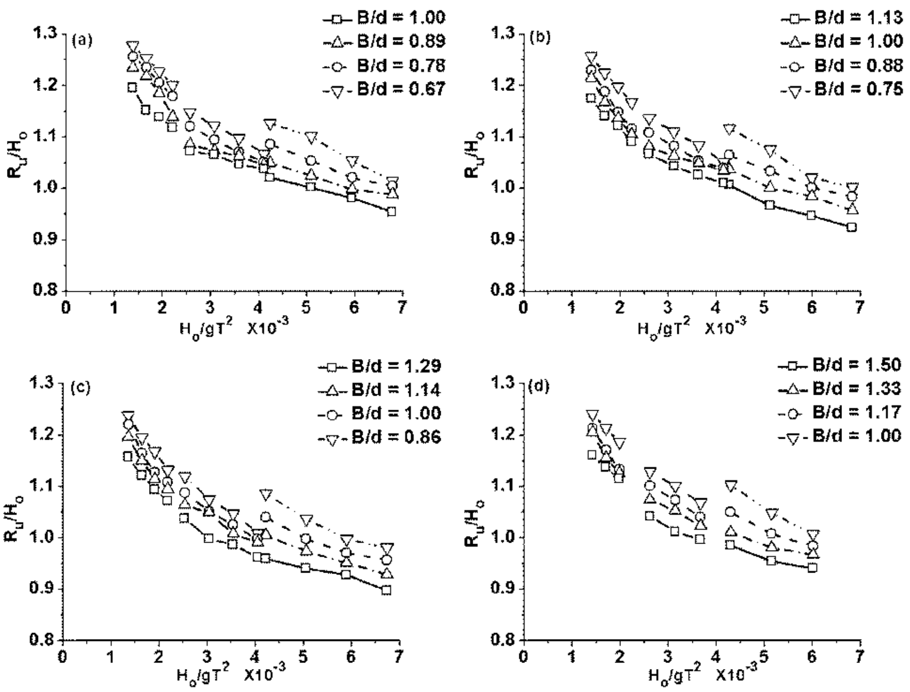


Fig. 4.9 (a) – (d) Influence of H_o/gT^2 on R_u/H_o for 25% reduction in armor weight and all water depths

Fig. 4.9 (a) – (d) and Fig. 4.10 (a) – (d) depict the influence of deepwater wave steepness (H_o/gT^2) on relative wave run-up (R_u/H_o) and relative wave run-down

(R_d/H_o) for 0.75W armor weight models in different water depths. It is observed that in all the cases wave run-up and run-down reduces with the increase in wave steepness. Further, in Fig. 4.9 it is observed that for longer period waves (H_o/gT^2 = 0.0013 – 0.00225) the trend line is steeper and the R_u/H_o values converge, irrespective of berm width, compared to shorter period waves (H_o/gT^2 = 0.00425 – 0.00675). In case of wave run-down also there is a similar steep decrease in R_d/H_o for longer period waves, but, the R_d/H_o values do not converge as in the case of R_u/H_o. As the berm width is increased, both R_u/H_o and R_d/H_o are found to decrease for all H_o/gT^2. This is because the wider berm width which provides large area for the wave energy dissipation.

For a water depth 0.45 m (Fig. 4.9 (a)), as berm width increases from 0.30 m to 0.35 m, 0.40 m and 0.45 m, the wave run-up decreases by 1% to 1.6%, 2.6% to 3.4% and 6% to 6.5% respectively. For a water depth 0.40 m (Fig. 4.9 (b)), as berm width increases from 0.30 m to 0.35 m, 0.40 m and 0.45 m, the wave run-up decreases by 1.8% to 2.1%, 3.3% to 4.4% and 6.5% to 7.8% respectively. For a water depth 0.35 m (Fig. 4.9 (c)), as berm width increases from 0.30 m to 0.35 m, 0.40 m and 0.45 m, the wave run-up decreases by 1.4% to 2.4%, 3.4% to 5.3% and 6.5% to 8.6% respectively. For a water depth 0.30 m (Fig. 4.9 (d)), as berm width increases from 0.30 m to 0.35 m, 0.40 m and 0.45 m, the wave run-up decreases by 2% to 2.4%, 2.9% to 4.1% and 6.4% to 6.6% respectively.

For a water depth 0.45 m (Fig. 4.10 (a)), as berm width increases from 0.30 m to 0.35 m, 0.40 m and 0.45 m, the wave run-down decreases by 4.5% to 7.5%, 8.6% to 12.7% and 19.5% to 17.4% respectively. For a water depth 0.40 m (Fig. 4.10 (b)), as berm width increases from 0.30 m to 0.35 m, 0.40 m and 0.45 m, the wave run-down decreases by 5.2% to 8.8%, 12.6% to 13.9% and 19.5% to 23.9% respectively. Further, For a water depth 0.35 m (Fig. 4.10 (c)), as berm width increases from 0.30 m to 0.35 m, 0.40 m and 0.45 m, the wave run-down decreases by 8.2%, 14.1% to 14.7% and 19.9% to 24.9% respectively. Similarly, For a water depth 0.30 m (Fig. 4.10 (d)), as berm width increases from 0.30 m to 0.35 m, 0.40 m and 0.45 m, the wave run-down decreases by 1.7% to 5.9%, 10.4% to 13.5% and 18.4% to 19.5% respectively.

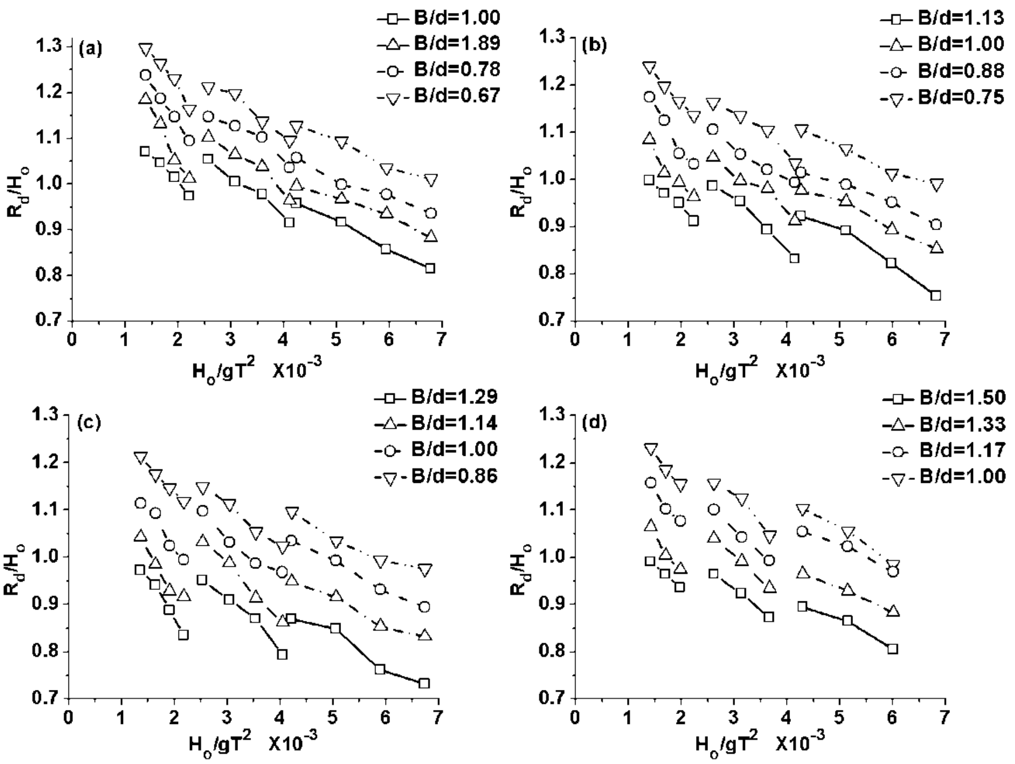


Fig. 4.10 (a) – (d) Influence of H_o/gT^2 on R_d/H_o for 25% reduction in armor weight and all water depths

Considering all the water depths and wave steepness, a maximum reduction in wave run-up and run-down of 8.6% and 24.9% respectively was observed in a 0.35 m depth of water for an increase in berm width from 0.30 m to 0.45 m.

4.5.3 Effect of change in thickness of primary layer on berm recession, wave run-up and run-down

The influence of deepwater water steepness (H_o/gT^2) on relative berm recession (R_{ec}/B) for different water depths and no. of primary layers (n) is shown in Fig. 4.11. A safe berm width (0.40 m) and safe reduction in armor weight (25%) are considered for investigation in this section. The thicknesses of primary layers are represented in terms of number of layers considered for calculating the thickness using Eq. 3.9. It is observed from the Fig 4.11, with the increase in H_o/gT^2, a similar increasing trend of R_{ec}/B is observed irrespective of thickness of primary armor.

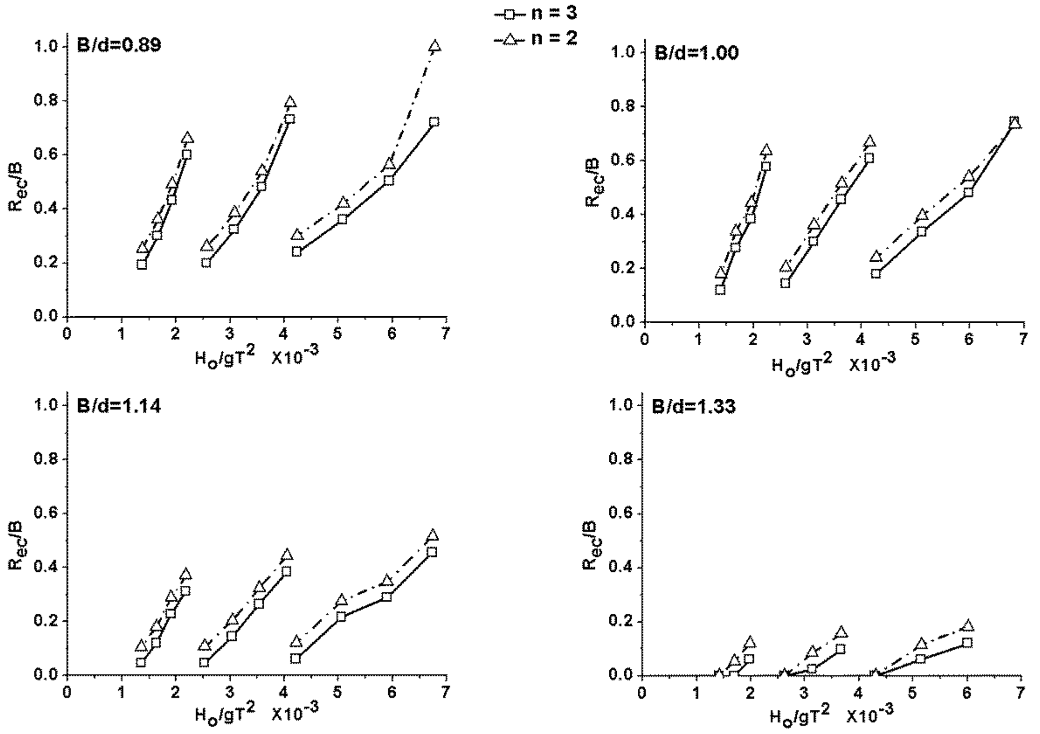


Fig. 4.11 Influence of H_o/gT^2 on R_{ec}/B for 0.40 m berm width and all water depths

From the Fig. 4.11 the variation in thickness of primary armor layer has a little effect on berm recession and it reduces with the increase in thickness of primary layer. A similar increasing trend of berm recession, as in the section 4.5.2, with the increase in wave steepness is observed in the Fig. 4.11. This is due to increasing energy of steeper waves. Also, a parallel trend for the both the thicknesses are observed for a particular B/d and all wave steepness.

For n = 2 and 3 it is observed that all the models are safe for almost all the considered wave conditions and water depths. In case of n= 2, for a B/d of 0.89 (d = 0.45 m) and H_o/gT^2 of 0.0064 (H= 0.16 m, T= 1.6 s) failure of structure is observed. For B/d = 1.00, 1.14 and 1.33 all the models are safe for the entire test conditions considered. Also, it is observed that with the increase in B/d there is reduction in berm recession and for the highest B/d of 1.33, a zero recession for wave height of 0.10 m for all wave periods (H_o/gT^2 = 0.0014, 0.0025, 0.0042) is observed for both the thicknesses.

There is a decrease in berm recession with the increase in thickness of primary layer as shown in the Fig. 4.11. This is because thicker layer of armor with more pores within the berm portion in the primary layer offers greater opportunity for dissipating more wave energy. This reduces the erosion of the berm and damage to the structure. Since, all the models were safe, except for extreme waves of 0.16 m and 1.6 s, with the lesser thickness it can be concluded that thickness of 2 is sufficient until the berm recession is less than the berm width.

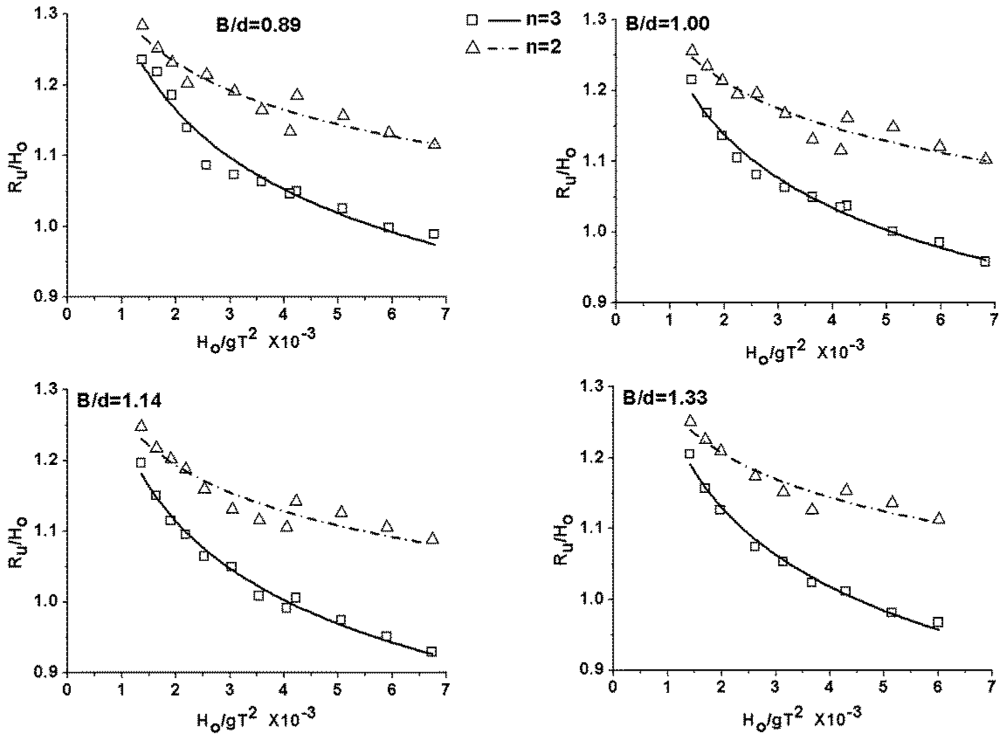


Fig. 4.12 Influence of H_o/gT^2 on R_u/H_o for 0.40 m berm width and all water depths

The influence of deepwater water steepness (H_o/gT^2) on relative wave run-up (R_u/H_o) for a constant berm width (0.40 m) and reduction in armor weight (25%) for different water depths and thickness of primary layers (n=2 and n=3) is presented in Fig. 4.12. From the figure it is clear that variation in thickness of primary armor layer has an effect on wave run-up and it reduces with the increase in thickness of primary layer. It is also observed that the trend line deviates away from each other as wave steepness increases. This is because of increased pore spaces for dissipation of energy of steeper waves in case of n = 3 compared to n = 2.

For a B/d of 0.89, as thickness of primary layer increases from 2 to 3, the wave run-up was reduced by 3.8% to 11.3% for the range of wave steepness considered. Similarly, for higher B/d of 1.00, 1.14 and 1.33, as 'n' increases from 2 to 3, the wave run-up decreases by 3.3% to 13.1%, 4.1% to 14.6% and 3.7% to 13.1% respectively.

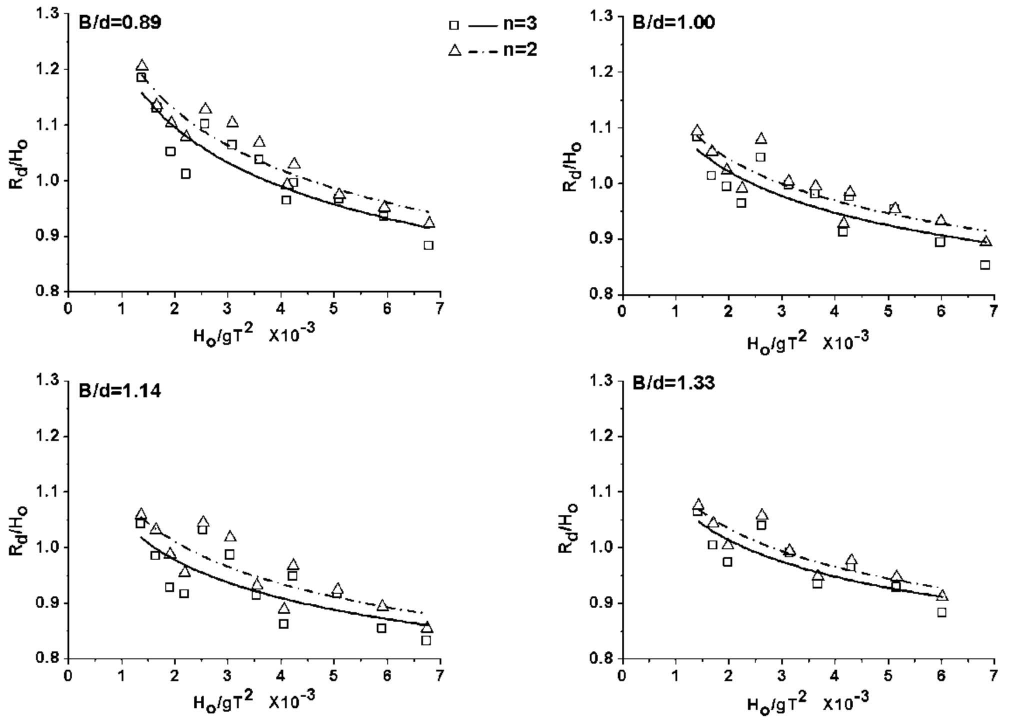


Fig. 4.13 Influence of H_o/gT^2 on R_d/H_o for 0.40 m berm width and all water depths

The effect of change in thickness of primary armor layer can be observed in Fig. 4.13. The influence of deepwater water steepness (H_o/gT^2) on relative wave run-down (R_d/H_o) for a constant berm width (0.45 m) and reduction in armor weight (25%) for different water depths and thickness of primary layers (n) is showed in the figure. It is clear that wave run-down is affected by the variation in thickness of primary armor layer and it reduces with the increase in thickness of primary layer. The decrease is not as significant as in the case of wave run-up. The trend lines in all the graphs are almost parallel to each other for the entire range of wave steepness, while, the trend lines of wave run-up (Fig. 4.12) showed a large variation as wave steepness increased.

For the entire range of wave steepness and B/d, as the thickness of primary layer is increased from 2 to 3, the wave run-down decreases by upto 4.6% which is insignificant compared to wave run-up which is upto 14.6%. The return flow of the wave run-down is not affected by the increased porosity of the armor layer. Hence, there is only small impact of variation in thickness of primary layer on wave run-down.

4.5.4 Influence of change in water depth on berm recession, wave run-up and run-down

From the section 4.5.3 it is observed that the model with 25% reduced armor weight and 3 no. of primary layer thickness for 0.40 m berm width is safe for all wave conditions and water depths. Further tests were conducted only for this condition and the results and analysis for the same are presented in all the subsequent sections.

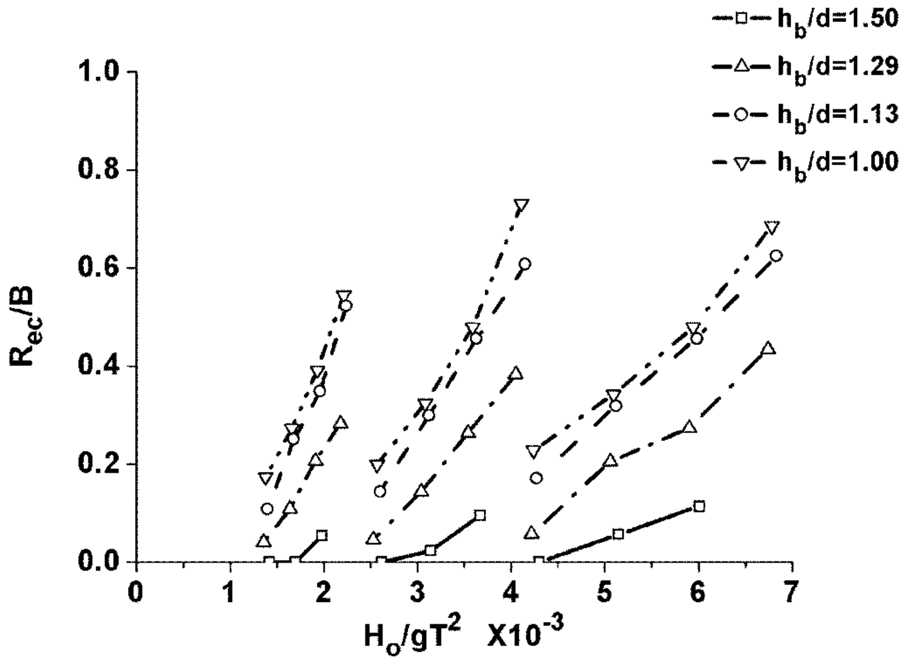


Fig. 4.14 Influence of H_o/gT^2 on R_{ec}/B for 0.40 m berm width and 25% reduction in armor weight

Fig. 4.14 depicts the effect of variation in water depth (expressed in terms of relative berm position, h_b/d) on relative berm recession (R_{ec}/B). The position of berm from the sea bed (h_b) was kept constant at a height of 0.45 m. The damage increases with an increase in depth of water and with a decreasing wave period. As observed in the previous sections 4.5.1 to 4.5.3, the R_{ec}/B increases with an increase in H_o/gT^2. For all the test conditions, the berm recession is less than berm width which indicates the structure is safe. High berm recession is observed when water level is equal to the berm position (i.e., h_b/d = 1) for all H_o/gT^2 considered. Berm recession is reduced as water level in front of the berm reduces and highest reduction of recession is observed for $h_b/d=1.50$. A maximum R_{ec}/B of 0.72 is observed for H_o/gT^2 of 0.0067 and water depth of 0.45 m ($h_b/d = 1.00$). As the h_b/d increases from 1.00 to 1.13, 1.29 and 1.50, the berm recession decreases by 8.8% to 37.5%, 58.1% to 76% and 76.1% to 100% respectively.

The high erosion of berm was observed when its level equals water level. This is because waves directly attack the upper slope and dislodge the armor units. Further, the return flow of the water pull down these dislodged armors along with it thus increasing erosion. But, as water level decreases the berm restricts the movement of the wave onto the upper reaches and hence, the return flow will also be less thus reducing the erosion. Even though the waves impinges directly on the lower slope for lower water depth ($h_b/d =1.50$), the energy within the wave is less and hence does not cause much damage to the structure. From the stability point of view it may be noted that when water depth is 0.35 m ($h_b/d = 1.29$) the reduction in berm recession is high. The large damage for higher water depths and shorter wave periods is because higher water depths sustain larger waves and short period waves repeatedly disturb the displaced armor units within a smaller time interval without allowing them to settle.

Fig. 4.15 shows the obtained relative wave run-up (R_u/H_o) as a function of the deepwater wave steepness (H_o/gT^2) for a constant berm width of 0.40 m and 25% reduced armor weight models. The water depth is varied from 0.30 m to 0.45 m ($h_b/d = 1.50 – 1.00$). The berm is placed at a constant height of 0.45 m from the sea bed. It is clearly observed that for a higher water depth of 0.45 m and 0.40 m ($h_b/d = 1.0, 1.13$), the wave run-up is high compared to other water depths. A lowest wave run-up is observed for a water depth of 0.35 m ($h_b/d = 1.29$). Wave run-up for 0.30 m water depth ($h_b/d = 1.50$) is more than 0.35 m depth. This is because for lowest water depth of 0.30 m, the effect of berm is not felt as the wave impinges directly on lower slope thus increasing wave run-up. Same is the case for higher water depth of 0.45 m where the water impinges on upper slope and hence, an increasing wave run-up. Further, a decrease in wave run-up with increasing H_o/gT^2 is also observed which is similar to the cases in the previous sections 4.5.1 to 4.5.3. Also, it can be observed that slope of trend line becomes gentle as H_o/gT^2 increases.

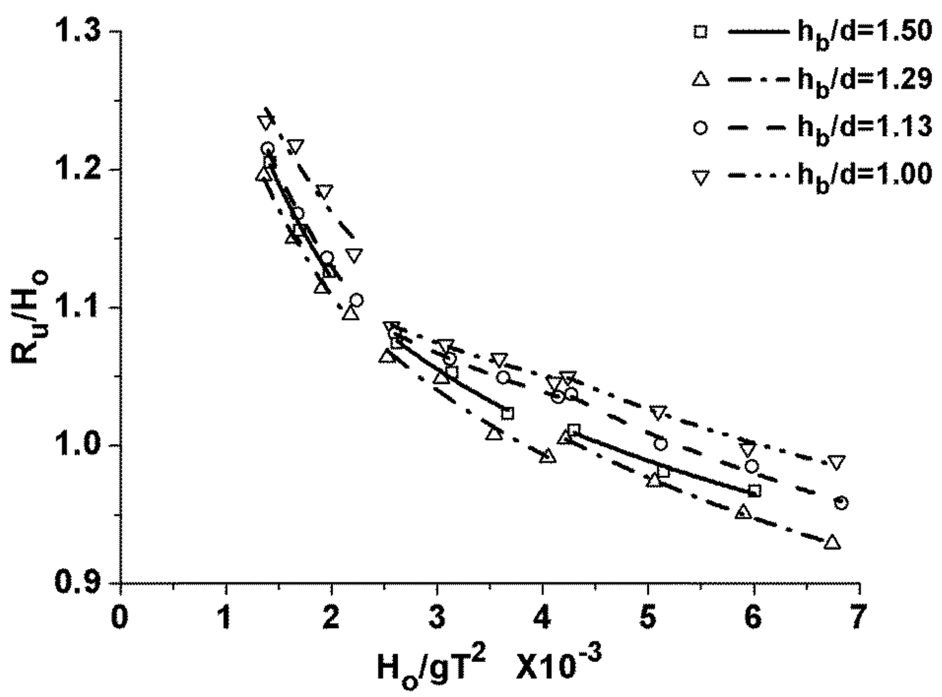


Fig. 4.15 Influence of H_o/gT^2 on R_u/H_o for 0.40 m berm width and all water depths

As h_b/d increases from 1.00 to 1.13, 1.29 and 1.50, the wave run-up decreases by 0.5% to 3.2%, 2.1% to 6.5% and 1.2% to 2.9% respectively for the entire range of wave steepness considered. It can be observed that the maximum reduction in R_u/H_o is 6.5% for h_b/d = 1.29. This implies that highest reduction in wave run-up is possible when h_b/d = 1.29 (d = 0.35m).

Fig. 4.16 shows the variation of relative wave run-down (R_d/H_o) as a function of the deepwater wave steepness (H_o/gT^2) for a constant berm width of 0.40 m and 25% reduced armor weight models. The water depth is varied from 0.30 m to 0.45 m (h_b/d = 1.50 – 1.00). The berm is placed at a constant height of 0.45 m from the sea bed. Wave run-down is high for highest water depth tested i.e., 0.45 m (h_b/d = 1.00) and lowest for a water depth of 0.35 m (h_b/d = 1.29) similar to that of wave run-up. It is also observed that as wave steepness increase the slope of trend line gets gentle for all the water depths considered. The steeper slope of lower H_o/gT^2 is because more energy is dissipated for longer period waves while for shorter period waves energy dissipation within the short interval between waves is less and hence, gentle trend line.

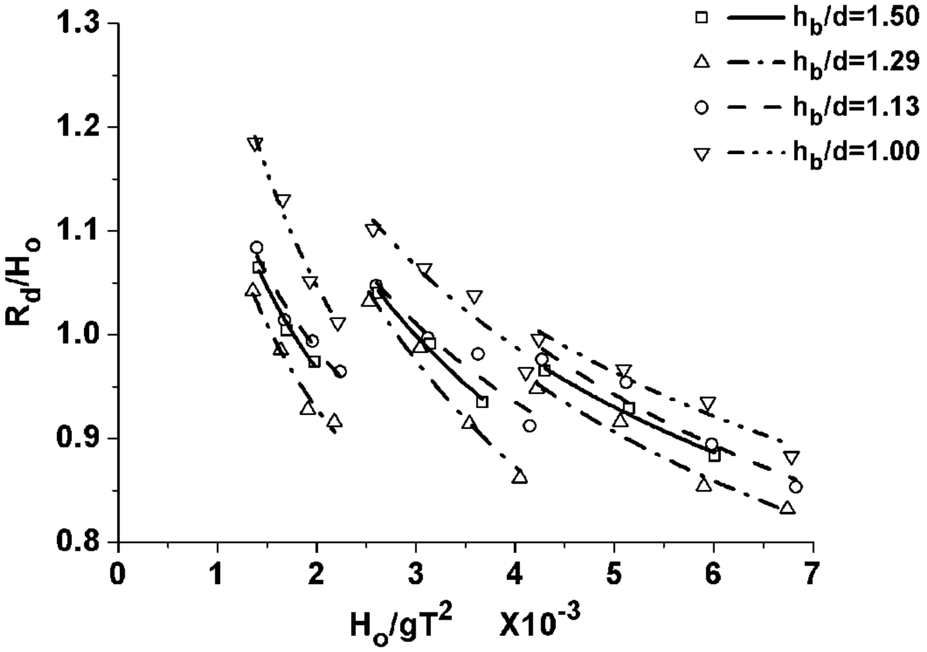


Fig. 4.16 Influence of H_o/gT^2 on R_d/H_o for 0.40 m berm width and all water depths

For all the water depths and wave steepness considered, as h_b/d increases from 1.00 to 1.13, 1.29 and 1.50, the R_d/H_o decreases by 4.6% to 9.3%, 9.5% to 13.7% and 5.9% to 11.3% respectively. It is observed that the maximum reduction in R_d/H_o is 13.7% for h_b/d = 1.29 which is higher compared to relative wave run-up where it was 6.5%. This implies that highest reduction in wave run-down is possible when h_b/d = 1.29 (d = 0.35m).

4.5.5 Influence of change in wave height and wave period on berm recession, wave run-up and run-down

The effect of equivalent surf similarity parameter (ξ_{eq}) on dimensionless berm recession (R_{ec}/B) for different wave heights and water depths can be observed in Fig. 4.17. The model with 25% reduced armor weight with 0.40 m berm width and 3 no. of primary armor layer thickness is considered to study the effect. Table 4.1 shows the deepwater surf similarity parameter (ξ_o) and equivalent surf similarity parameter (ξ_{eq}) calculated using the method explained in CEM 2006 (refer Fig. 2.17), for different wave heights and wave periods considered for the study.

$$\xi_o = \frac{\tan\alpha}{\sqrt{H_o/L_o}} \quad \text{...} \quad (4.1)$$

$$\xi_{eq} = \frac{\tan\alpha_{eq}}{\sqrt{H_o/L_o}} \quad \text{...} \quad (4.2)$$

Table 4.1 Equivalent surf similarity parameter for 0.40 m berm width model

H (m)	T (s)	α_{eq}	ξ_o	ξ_{eq}
0.10	1.6	16^0	4.070	1.743
0.12	1.6	16^0	3.677	1.591
0.14	1.6	16^0	3.439	1.473
0.16	1.6	16^0	3.202	1.378
0.10	2.0	16^0	5.092	2.229
0.12	2.0	16^0	4.674	2.035
0.14	2.0	16^0	4.383	1.884
0.16	2.0	16^0	4.085	1.762
0.10	2.6	16^0	7.087	3.021
0.12	2.6	16^0	6.507	2.758
0.14	2.6	16^0	6.006	2.554
0.16	2.6	16^0	5.658	2.389

Fig. 4.17 shows the variation in the berm recession as the equivalent surf similarity parameter changes from 1.3 to 2.9 for different water depths of 0.35 m, 0.40 m and 0.45 m (B/d= 1.14, 1.00 and 0.89). It is observed that with the increase in ξ_{eq}, the berm recession decreases, while, with the increase in N_s, berm recession increases. Also, with the increase in B/d ratio the berm recession is found to decrease for all the test conditions. For B/d = 0.89, 1.00 and 1.14, the berm recession varies from 0.192 to 0.732, 0.12 to 0.657 and 0.046 to 0.456 respectively. With the increase in B/d from 0.89 to 1.00 and 1.14, there is 8.9% to 37.5% and 36.7% to 76.0% reduction in berm recession respectively for all the test conditions.

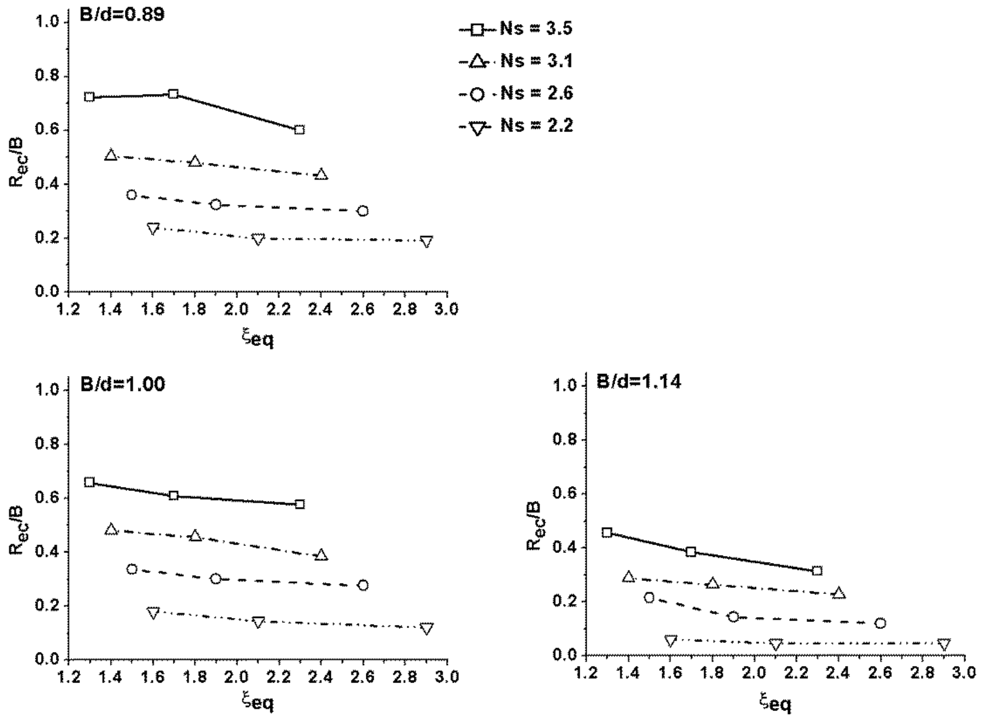


Fig. 4.17 Influence of ξ_{eq} on R_{ec}/B for 0.40 m berm width and 25% reduction in armor weight

For a particular B/d of 0.89, with the increase in ξ_{eq} from 1.6 to 2.9, the R_{ec}/B decreases by 14.3% to 20%. Similarly, for a particular B/d of 0.89, as N_s decreases from 3.5 to 2.2, the R_{ec}/B also decreases by 66.7% to 72.8%. Comparing the percentage decrease with respect to wave period (ξ_{eq}) and wave height (N_s), it can be concluded that the effect of wave period is less on berm recession. The same observation was also expressed by Moghim et al. (2011).

According to Bruun and Gunbak (1976), the failure of breakwater is caused by combinations of buoyancy, inertia and drag forces supported by the effect of hydrostatic pressure from the core. These forces all seem to reach their maximum value for lowest down rush which occurs at resonance. Bruun and Gunbak (1976) observed that, this condition was reached for $2.0 < \xi_o < 3.0$. In the present study, models tested with wave period of 1.6 s (ξ_{eq} = 1.2 – 1.6) have shown greater damage when compared to the other wave periods studied. This may be due to the nature of wave breaking, which is of collapsing type when period is 1.6 s and corresponding value of ξ_o is 3.2 to 4.0 and ξ_{eq} is 1.37 to 1.74. The recession was found decreasing as the wave period increased from 1.6 s to 2.6 s. In model study, during such condition, it is found that wave run-up is maximum and run-down is relatively high and armor units are disturbed, dislodged and lifted by run-up and pulled down by the run-down as stated by Ahrens (1970).

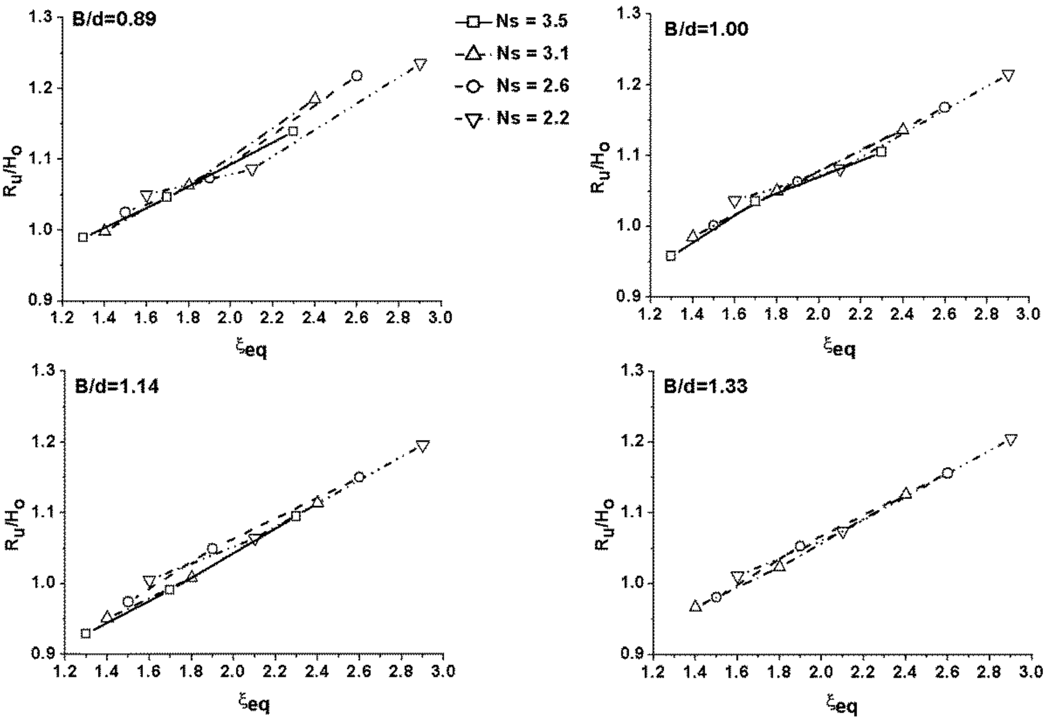


Fig. 4.18 Influence of ξ_{eq} on R_u/H_o for 0.40 m berm width and 25% reduction in armor weight

The effect of wave period and height on wave run-up is observed in Fig. 4.18. The graphs are plotted for a constant berm width of 0.40 m and 25% reduced armor weight for different water depths. A steep increase in wave run-up with increase in surf similarity parameter (ξ_{eq}) can be observed in the graphs for all stability numbers. From the figure, it is clear that R_u/H_o is linearly proportional to the ξ_{eq} for any B/d. For any selected B/d, it can be seen that, higher wave periods cause increased R_u/H_o. This observation confirms the similar trend demonstrated in Fig. 4.15 of section 4.4.4. It is also demonstrated in Fig. 4.18, that B/d has little influence on R_u/H_o Vs ξ_{eq}, since, the slope of the graphs are almost similar. Also, the relative wave run-up increases with the increase in stability number which can also be noticed very clearly from the graphs. This graph makes a clear indication of dependence of wave run-up on wave period and wave height.

For a B/d of 1.00, as N_s increases from 2.2 to 3.6, the R_u/H_o decreases by 3.6% to 10%. Further, for a B/d of =1.00 and N_s of 2.2, as ξ_{eq} increases, R_u/H_o also increases by 4% to 14%.

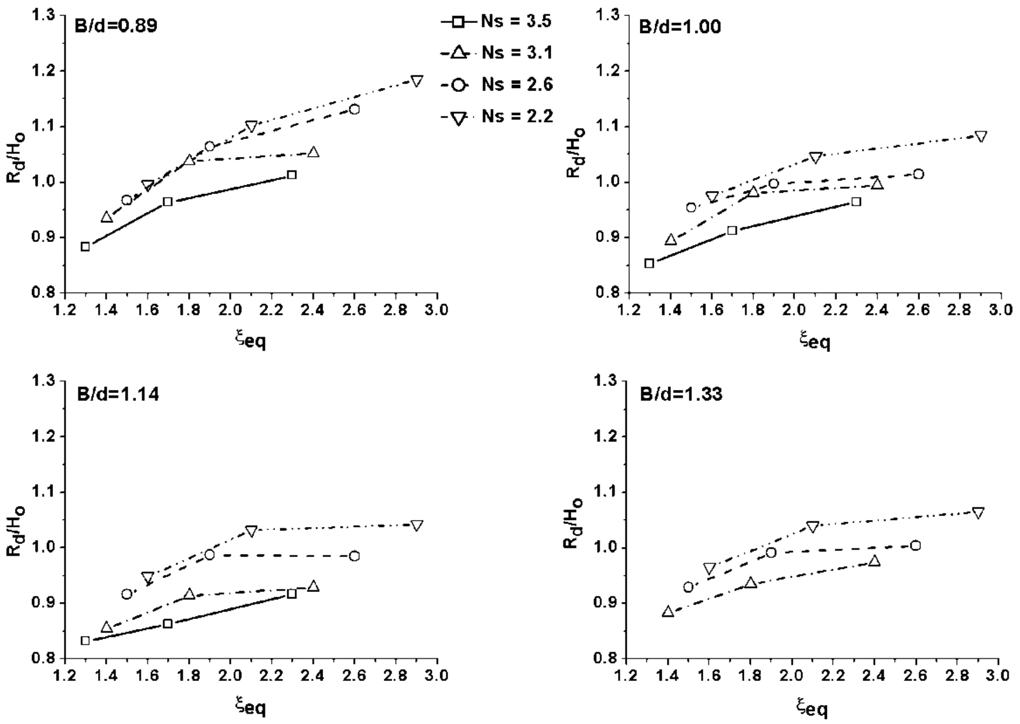


Fig. 4.19 Influence of ξ_{eq} on R_d/H_o for 0.40 m berm width and 25% reduction in armor weight

The effect of wave period and height on wave run-down can be observed in Fig. 4.19. The graphs are plotted for a constant berm width of 0.40 m and 25% reduced armor weight for different water depths. An increase in wave run-down with increase in surf similarity parameter (ξ_{eq}) can be observed in the graphs for all stability numbers. It is also demonstrated in Fig. 4.19 that B/d has some influence on R_d/H_o Vs ξ_{eq}, since, with the increase in B/d the trend line starts becoming horizontal with increased ξ_{eq}. Also, the wave run-down increases with the increase in stability number which can also be noticed very clearly from the graphs. This graph makes a clear indication of dependence of wave run-down on wave period and wave height.

For a B/d of 1.00, as N_s increases from 2.2 to 3.6, the R_d/H_o decreases by 2.3% to 14.4%. Further, for a B/d of =1.00 and N_s of 2.2, as ξ_{eq} increases, R_d/H_o also increases by 6.7% to 10%.

4.5.6 Influence of storm duration on berm recession

Figs. 4.20 to 4.23 illustrate the influence of storm duration on dimensionless berm recession (R_{ec}/B) for different deepwater wave steepness and water depths. The effect is shown for 25% reduced armor weight with 0.40 m berm model for 3 layers of primary armor. It is observed that as storm duration increases the berm recession also increases. For all water depths, wave periods and lower wave height (H = 0.10 m) the berm is almost stabilized and recession was found to be nearly constant after 2000 waves (4.44 hrs to 7.22 hrs). The legend depicting the ‘dash’, ‘dotted’ and ‘solid’ lines represent 1.6, 2.0 and 2.6 s wave period respectively in all the figures.

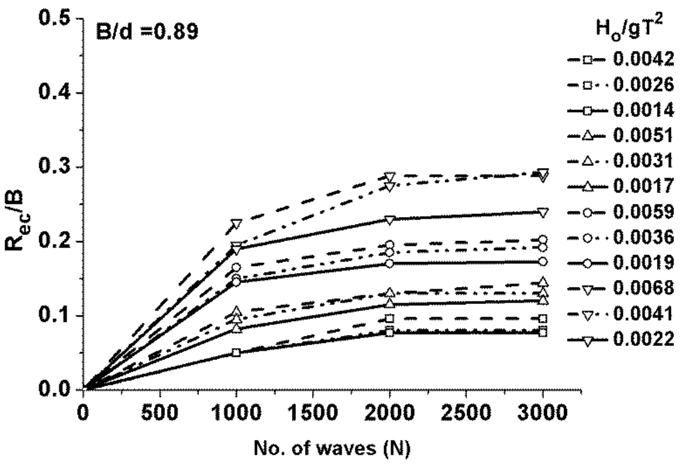


Fig. 4.20 Influence of no. of waves on R_{ec}/B for 0.40 m berm width and 0.45 m water depth

The effect of storm duration on dimensionless berm recession for a water depth of 0.45 m is observed in Fig. 4.20. It is clear from the graph that most of the settlement and movement of the armor takes place within 2000 waves. From the figure it is observed that as wave height increases berm recession increases and it decreases with the increase in wave period. 90% of the berm recession occurs within 2000 waves (4.44 hrs to 7.22 hrs) and the remaining upto 3000 waves (6.67 hrs to 10.83 hrs) for all the considered wave conditions. This is as suggested by many researchers (Van der Meer and Pilarczyk 1984, Hall 1984, Hegde 1996, Moghim et al. 2011).

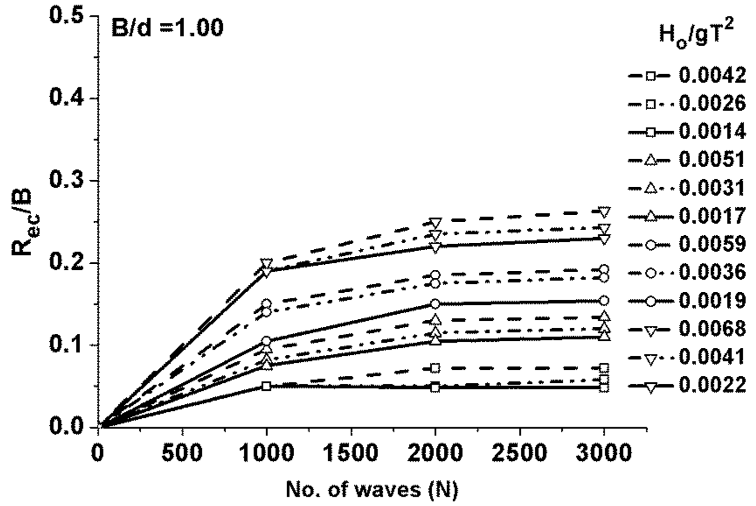


Fig. 4.21 Influence of no. of waves on R_{ec}/B for 0.40 m berm width and 0.40 m water depth

Similarly, from Fig. 4.21, the increase in berm recession with increasing wave height and decreasing wave period can be observed. Further, 90% of the berm recession occurred within 2000 waves for higher wave heights (H = 0.14 m, 0.16 m) and within 1000 waves (2.22 hrs to 3.61 hrs) for lower wave heights (H = 0.10 m, 0.12 m).

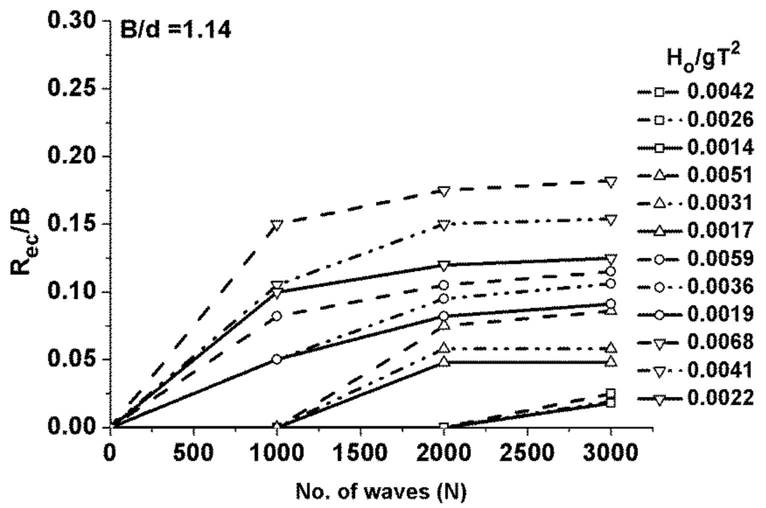


Fig. 4.22 Influence of no. of waves on R_{ec}/B for 0.40 m berm width and 0.35 m water depth

In Fig. 4.22 it can be clearly observed that for a wave height of 0.10 m (H_o/gT^2 = 0.0042, 0.0026, 0.0014) there was no berm recession upto 2000 waves for all wave periods. Similarly, for a wave height of 0.12 m (H_o/gT^2 = 0.0051, 0.0031, 0.0017), no berm recession was observed for initial 1000 waves for all wave periods. For waves of 0.14 m (H_o/gT^2 = 0.0059, 0.0036, 0.0019) and 0.16 m (H_o/gT^2 = 0.0068, 0.0041, 0.0022), berm recession was increasing for the entire storm duration.

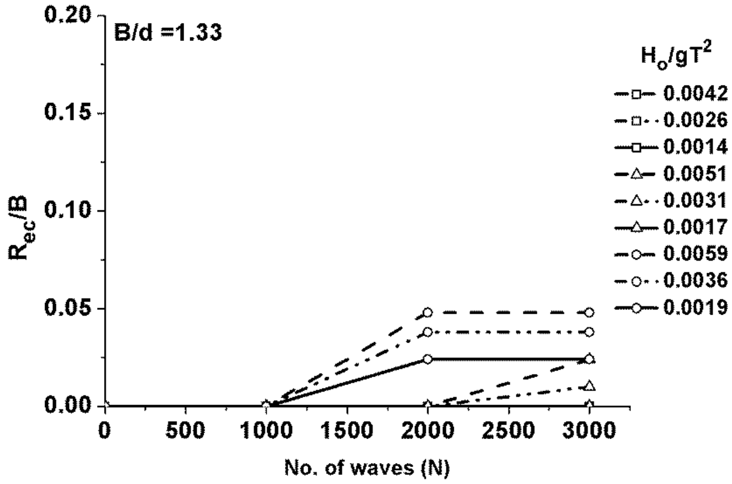


Fig. 4.23 Influence of no. of waves on R_{ec}/B for 0.40 m berm width and 0.30 m water depth

For a water depth of 0.30 m (Fig. 4.23) no berm recession occurred for waves of 0.10 m and all wave periods (H_o/gT^2 = 0.0042, 0.0026, 0.0014) and for 0.12 m wave with 2.6 s period (H_o/gT^2 = 0.0017) for the entire storm duration. For 0.14 m wave and all wave periods (H_o/gT^2 = 0.0059, 0.0036, 0.0019), for initial 1000 waves no recession of berm was observed and upto 2000 waves, the reshaping of structure was observed after which no further erosion was observed.

The decrease in rate of increase in berm recession with the increase in storm duration is due the reshaping of the breakwater structure into an equilibrium profile during the wave attack. Once equilibrium profile is attained even with further attack of waves, no further erosion of berm will takes place. This makes storm duration an important parameter in case of berm breakwater which helps in knowing after how much storm/cyclone/depression in sea, an equilibrium profile is achieved for a particular wave condition.

4.6 EQUATIONS DEVELOPED FROM THE PRESENT STUDY

All the test data for different configurations of berm breakwater were combined into suitable dimensionless terms and are represented in Fig. 4.24. The figure shows the experimental data for 540 test runs and the best fit curve with a correlation coefficient of 0.8251. The curve is fitted using M.S. Excel sheet.

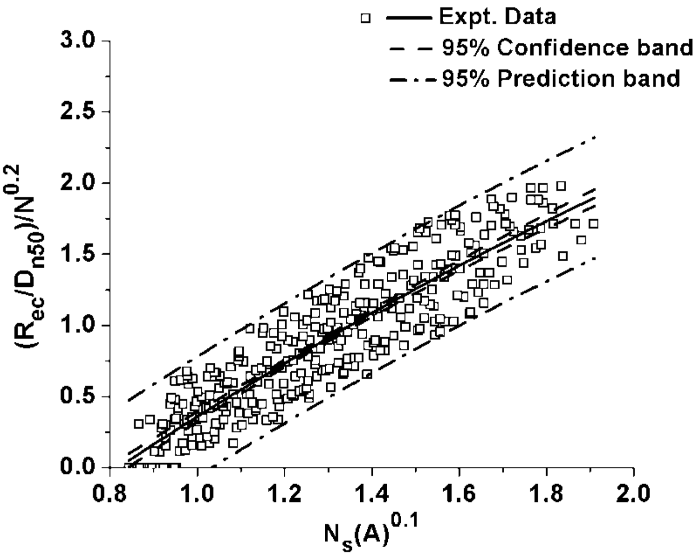


Fig. 4.24 Stability equation for berm breakwater

From Fig. 4.24, the equation for berm recession in statically stable reshaped berm breakwater is derived as:

$$\frac{R_{ec}/D_{n50}}{N^{0.2}} = N_s[A]^{0.1} \text{--} \quad (7.1)$$

$$\text{Where, } A = \left[\frac{(H_o/gT^2)*10^{-3}}{n*(B/d)*(h_b/d)}\right] \text{------------------------} \quad (7.2)$$

Fig. 4.25 shows the comparison between the measured berm recession and the calculated berm recession. A good R^2 of 0.821 is obtained.

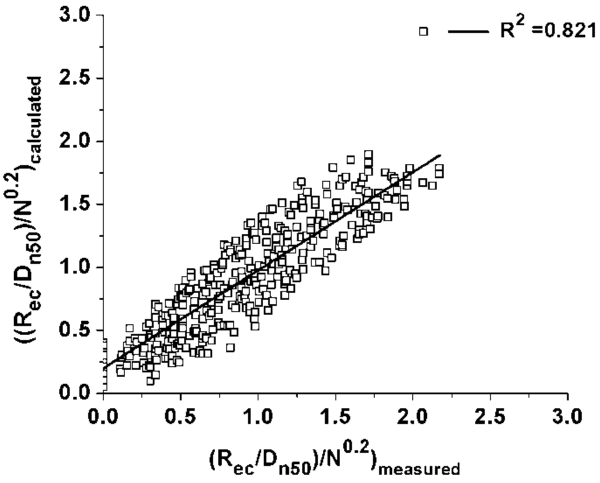


Fig. 4.25 Comparison of measured and calculated berm recession

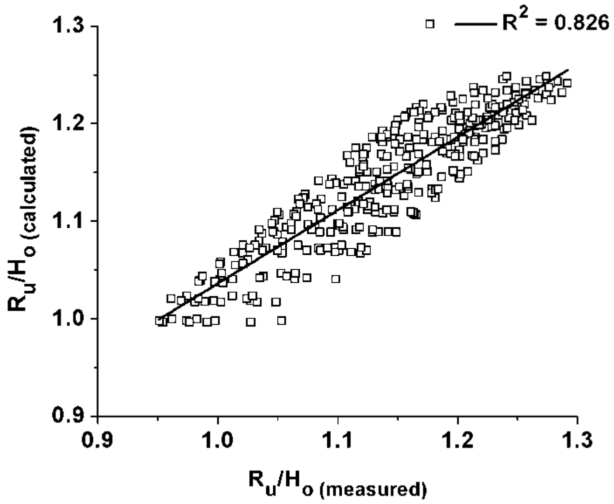


Fig. 4.26 Comparison of measured and calculated wave run-up

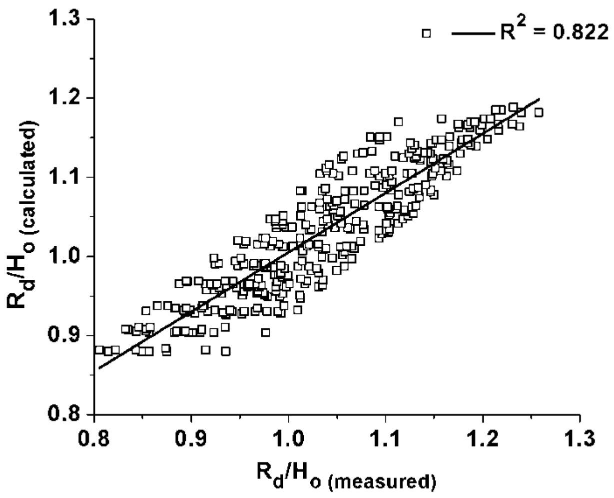


Fig. 4.27 Comparison of measured and calculated wave run-down

Similarly equations for prediction of wave run-up and run-down are also developed which are given as:

$$\frac{R_u}{H_o} = \frac{2.449\xi_{eq}}{1 + 1.681\xi_{eq}} \text{------------------------} \quad (7.3)$$

$$\frac{R_d}{H_o} = \frac{1.680\xi_{eq}}{1 + 1.133\xi_{eq}} \text{------------------------} \quad (7.4)$$

Fig. 4.26 shows the comparison between the measured wave run-up and the calculated wave run-up using Eq. 7.3. A better R^2 of 0.821 is obtained. Similarly, Fig. 4.27 depicts the comparison between the measured wave run-down and the calculated wave run-down using Eq. 7.4. A better R^2 of 0.822 is obtained.

4.7 OPTIMUM BERM BREAKWATER CONFIGURATION

Considering the test results, an optimum berm breakwater design is evolved. This consists of a reduced armor cube weight of 79.5 g (≈0.75W) and lower crest height of 0.6 m (i.e. 14.28% of 0.7 m) with a crest width of 0.1 m. It also consists of a horizontal berm of 0.40 m width at a height of 0.45 m from sea bed. A slope of 1V:1.5H in the remaining upper and lower faces of the breakwater is provided. A minimum thickness of 2 primary armor layers can be provided as it is stable for most of the test conditions except for extreme waves of 0.16 m and 1.6 s. This model is tested for the similar wave characteristics as in the earlier investigations (Refer section 4.5.3). It is found that the breakwater is totally safe with $R_{ec}/B < 1$ with no exposure of secondary layer for the entire range of test parameters.

4.8 COMPARISON OF PRESENT BREAKWATER MODEL WITH OTHER TYPES OF BREAKWATER

4.8.1 Comparison of wave run-up and wave run-down with conventional breakwater

Fig. 4.28 and 4.29 shows the comparison of wave run-up and run-down for the present model and a conventional uniform slope breakwater, both, armored with concrete cubes. The berm breakwater model is made of 25% reduced weight armor units and has a 0.45 m berm with two layers of primary armor. The conventional breakwater consists of design weight armor units placed in three layers in the primary layer. Both breakwaters are constructed with a same sea side slope of 1:1.5.

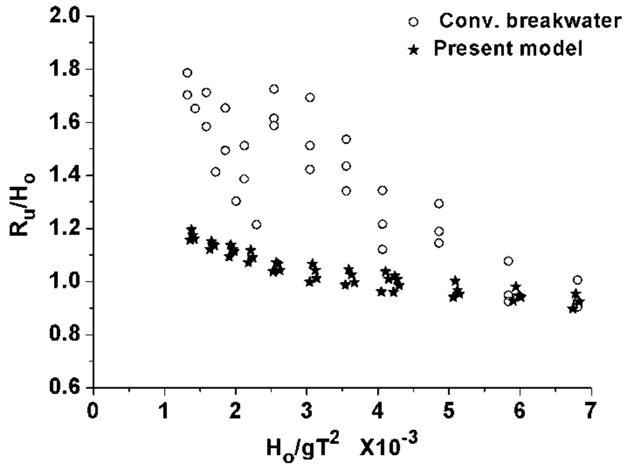


Fig. 4.28 Comparison of wave run-up between conventional breakwater and present model

From Fig. 4.28 it is clear that wave run-up is significantly decreased for berm breakwater as compared to conventional breakwater. This reduction in wave run-up can be attributed to the presence of berm which breaks the wave thus reducing run-up. For the range of wave steepness and water depths considered, the relative wave run-up in berm breakwater decreases by 1% to 34%.

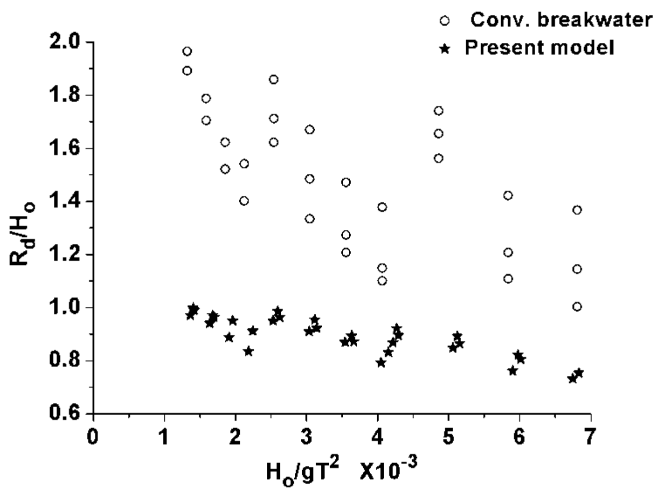


Fig. 4.29 Comparison of wave run-down between conventional breakwater and present model

Fig. 4.29 shows the comparison of wave run-down and even in this case it is clear that wave run-down is again significantly decreased for berm breakwater compared to conventional breakwater. For the range of water depths and wave steepness considered, relative wave run-down in berm breakwater decreases by 27% to 49% compared to conventional breakwater.

4.8.2 Comparison of berm recession, wave run-up and wave run-down with berm breakwater armored with natural stones

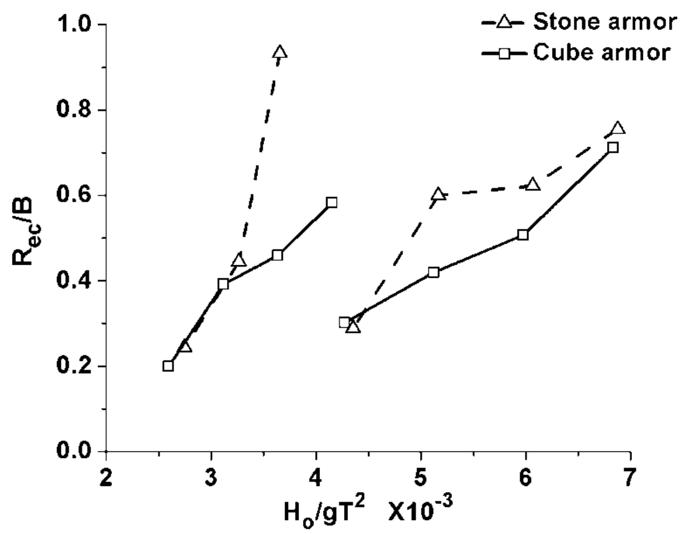


Fig. 4.30 Comparison of berm recession between berm breakwaters armored with cubes and stones

The comparison of relative berm recession (R_{ec}/B) between berm breakwaters armored with cubes and stones are shown in Fig. 4.30. Both the berm breakwater are built with a constant berm of 0.45 m, 40% reduced armor weight with three layers of primary armor. The results of the stone armored breakwater were collected from previous studies carried out by Rao (2009). It is clear from the graph that, for a water depth of 0.40 m (B/d = 1.13), cube armored breakwater has lesser berm recession compared to stone armored breakwater. For lower wave height a lower recession can be observed in case of stone armored breakwater but with the increase in wave height the berm recession also increases. The reason for lower berm recession is due to the size of the armor units in primary layer. In stone armored breakwater a range of weights are considered while in cube armored a single weight of the armor unit will be used. This affects the porosity of the layer thus increasing berm recession. For the range of wave steepness considered, R_{ec}/B in cube armored berm breakwater decreases by 18% to 51%.

In addition these results are also tabulated in Table 4.2 to get a clear idea of reduction or increase when cube armored berm breakwater is compared with cube armored conventional breakwater and stone armored berm breakwater.

Table 4.2 Comparison of results of present study with conventional cube armored and stone armored berm breakwaters

Parameter	**CCBW***	**CBBW# (B = 0.45 m)**			**SBBW$ (B = 0.45 m)**	
		25%	**40%**	**50%**	**30%**	**40%**
Max. R_u/H_o	1.785	1.195	1.215	1.248	1.177	1.284
% increase in R_u/H_o	49.4	--	1.7	4.4	1.5 (decrease)	7.5
Max. R_d/H_o	1.965	1.071	1.177	1.236	1.195	1.286
% increase in R_d/H_o	83.5	--	9.9	15.4	11.6	20.1
Max. R_{ec}/B	NA[1]	0.654	1.00	1.00	1.00	1.00
% increase in R_{ec}/B		--	52.9	52.9	52.9	52.9

* – CCBW – Conventional Cube armored Breakwater

– CBBW – Cube armored Berm Breakwater

$ – SBBW – Stone armored Berm Breakwater

NA[1] – Berm recession is not applicable in case of conventional cube armored breakwater

4.9 COMPARISON OF EXPERIMENTAL BERM RECESSION WITH DIFFERENT EQUATIONS

The berm recession obtained from the present experimental studies is compared with the berm recession obtained from different equations (Torum 1998, Torum et al. 2003, Andersen and Burcharth 2010, Moghim et al. 2011 and Shekari and Shafieefar 2013). Ranges of variables of each equation are taken into account while comparing the predicted values with the present experimental results.

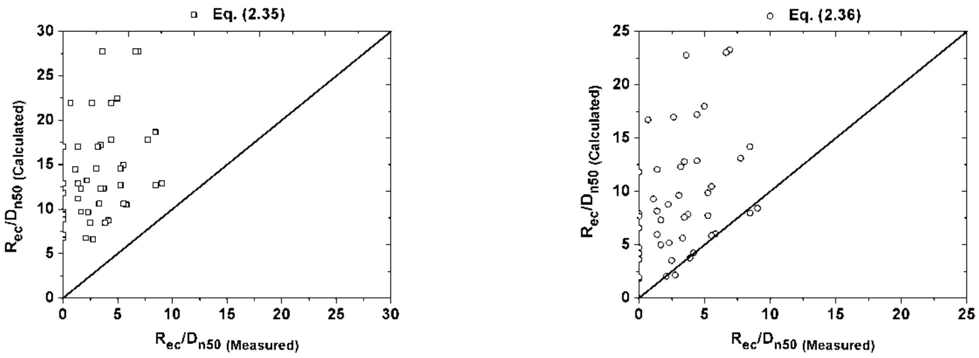


Fig. 4.31 Comparison of experimental results with Eq. 2.35 and Eq. 2.36

It is clear from the comparison shown in Fig. 4.31 between the experimental results and Eq. 2.35 and 2.36 that the equations over estimate the berm recession values. Though the ranges of variables for Eq. 2.36 do not match with the present study, the comparison shows that there is some agreement between the results.

Eq. 2.39 given by Andersen and Burcharth (2010) also over estimate the berm recession values compared to measured values (Fig. 4.32). Similarly, Eq. 2.41 and Eq. 2.43 also over predict the berm recession values as shown in Fig. 4.33.

The over estimation of the values by all the equations can be related to the conditions under which those equations were developed. All the equations considered here were developed for stone armored homogenous berm breakwater which is different from the berm breakwater of the present study.

Eqs. 2.35, 2.36, 2.39, 2.41 and 2.43 over estimate the R_{ec}/D_{n50} values by upto 77.3%, 65.4%, 77.9%, 82.5% and 73.1% respectively compared to present study.

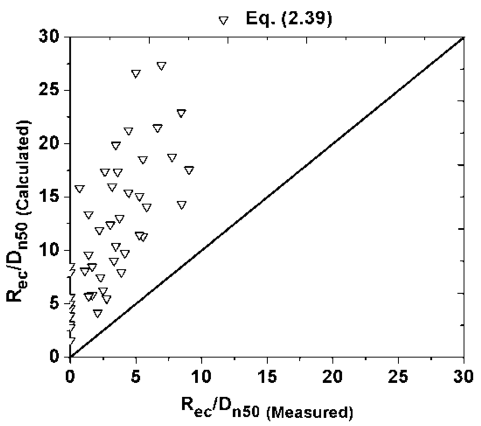


Fig. 4.32 Comparison of experimental results with Eq. 2.39

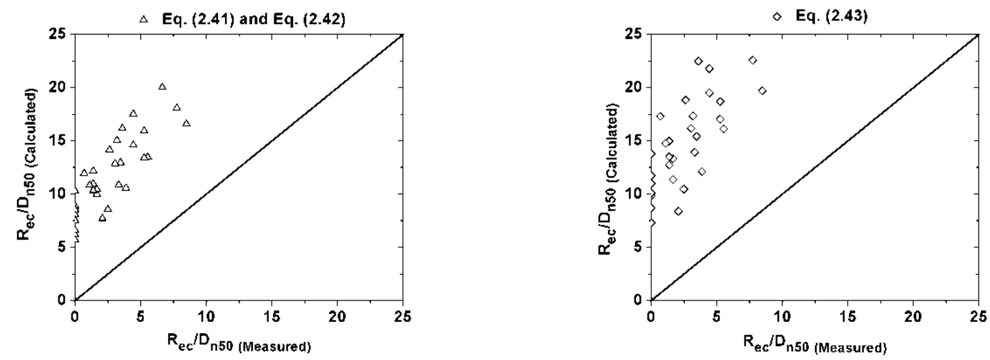


Fig. 4.33 Comparison of experimental results with Eq. 2.41 and Eq. 2.43

4.10 COMPARISON OF PRESENT EXPERIMENTAL WAVE RUN-UP WITH EQUATION 2.44

Figs. 4.34 and 4.35 show the comparison of present experimental results with the Eq. 2.44 (De Waal and Van der Meer 1992) which takes into account the influence of the berm width and water depth (see section 2.8). The equivalent surf similarity (ξ_{eq}) parameter varies with berm widths. Hence, there are differences in estimated values of wave run-up for different berm widths.

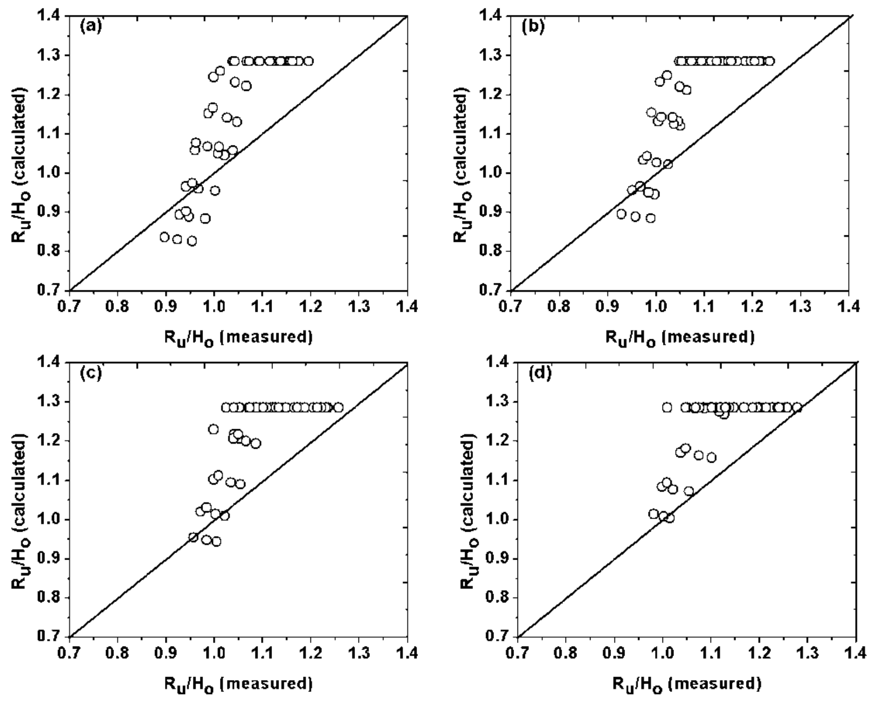


Fig. 4.34 Comparison of experimental wave run-up with Eq. 2.44 for shallow water condition

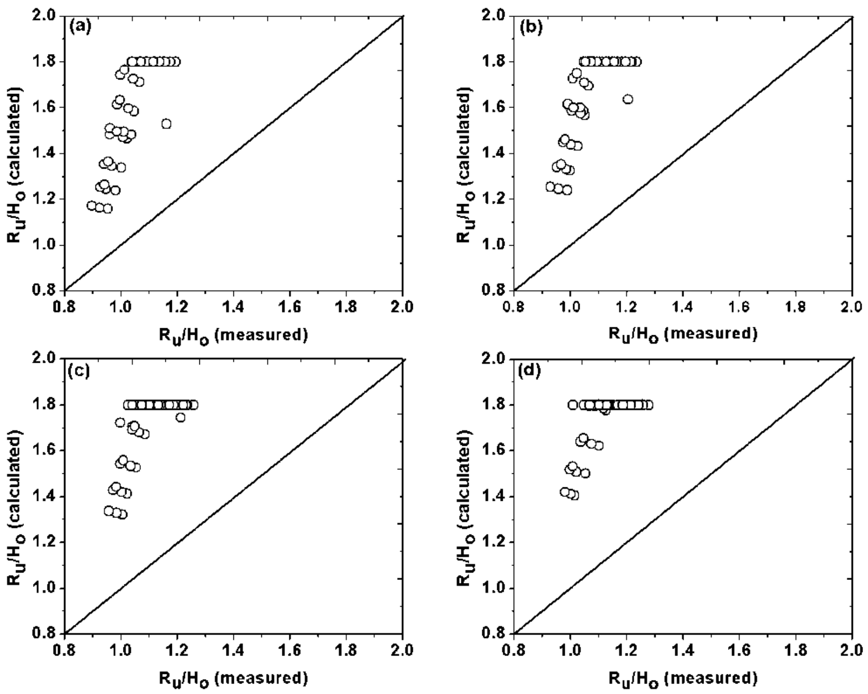


Fig. 4.35 Comparison of experimental wave run-up with Eq. 2.44 for deepwater condition

In the Fig. 4.34, the data related to the equations are calculated corresponding to shallow water condition, where γ_h, reduction factor for influence of shallow water conditions is taken as 0.714. From the Fig. 4.34 it is observed that the prediction of wave run-up is over-estimated in most of the cases. Also, as berm width decreases, it can be observed that more and more points starts accumulating at a R_u/H_o value of 1.3. The equation estimates a constant R_u/H_o of 1.3 for $\xi_{eq} \geq 2.0$ where as the experimental values vary.

In the Fig. 4.35, the data related to the equations are calculated corresponding to deep water condition where $\gamma_h = 1$. In this case also the wave run-up is over estimated for all the conditions considered.

4.11 COST ANALYSIS

The cost analysis of the prototype berm breakwater (details of which are given in Appendix II) is undertaken. It shows that the cube armored berm breakwater is about 8% and 4% economical than the conventional cube armored breakwater and stone armored berm breakwater (depending on site condition), respectively, both designed for same operating conditions.

4.12 SUMMARY

The results of various tests conducted on reshaped cube armored berm breakwater with different sea state and structural parameters were presented in this chapter. The results obtained are discussed in detail providing relevant details for various phenomena that occurred during the interaction of the wave with the structure.

It is observed that reduction in armor weight had a serious impact on stability and damage is found increasing with decrease in armor weight. Berm width also had some influence in reducing berm recession and it is found that increasing berm width greatly reduces berm recession. The thickness of primary armor layer also has its role in reducing berm recession and with increased thickness, higher reduction could be achieved. The water depth in front of the structure has a serious impact on berm recession and with the reduction in water depth, the berm recession also decreased. It is observed that even though decrease in wave period increased berm recession its effect compared to wave height is less which incurred higher recession. Finally, the storm duration had a great influence, since; this was the important factor which reshaped the structure.

The impact of various parameters on wave run-up and run-down is also studied. It could be noted that many of the parameters had no serious impact on run-up or run-down. The reduction in armor weight, increase in thickness of primary layer, water depth fluctuations had no major impact in reducing wave run-up or run-down. The increasing berm width and change in wave parameters has some notable impact. It is observed that 0.35 m depth of water (h_b/d = 1.29) had the lowest wave run-up and run-down. The summary of all tests conducted during the study is tabulated in Table 4.3 showing the maximum R_{ec}/B, R_u/H_o and R_d/H_o of all the test models.

Considering all the parameters and the obtained results, it is found that 25% reduction in armor weight with 0.40 m berm width and 3 no. of primary armor layers is safe for the entire conditions considered during the study. However, for most of the wave climate (excluding extreme waves of 0.16 m height and 1.6 s period) primary layer with 2 armor thickness is safe with a berm width of 0.40 m. In terms of safety as well as economy 25% reduction in armor weight with 0.40 m berm width and 2 no. of primary layer is cheaper compared to the entire models studied.

Table 4.3 Summary of results

Parameters	**B (m)**	**Design armor weight (W)**	**0.75W**		**0.6W**		**0.5W**
		n = 2	**n = 2**	**n = 3**	**n = 3**	**n = 4**	**n = 4**
Max. R_u/H_o for all depths	**0.45**	NA[1]	1.262	1.195	1.215	1.202	1.248
	0.40	NA[1]	1.284	1.235	NA[3]	NA[3]	NA[3]
	0.35	1.274	1.305	1.257	NA[3]	NA[3]	NA[3]
	0.30	1.292	NA[2]	1.278	NA[3]	NA[3]	NA[3]
Max. R_d/H_o for all depths	**0.45**	NA[1]	1.146	1.071	1.177	1.124	1.236
	0.40	NA[1]	1.205	1.185	NA[3]	NA[3]	NA[3]
	0.35	1.274	1.267	1.238	NA[3]	NA[3]	NA[3]
	0.30	1.297	NA[2]	1.297	NA[3]	NA[3]	NA[3]
Max. R_{ec}/B for all depths	**0.45**	NA[1]	0.78 (S)	0.65 (S)	1.00 (F)	1.00 (F)	1.00 (F)
	0.40	NA[1]	1.00 (F)	0.76 (S)	NA[3]	NA[3]	NA[3]
	0.35	0.80 (S)	1.00 (F)	1.00 (F)	NA[3]	NA[3]	NA[3]
	0.30	1.00 (F)	NA[2]	1.00 (F)	NA[3]	NA[3]	NA[3]

NA[1] – Not applicable as the berm breakwater with design armor weight (i.e., without reduction in weight) and 0.35 m berm width was safe for all test conditions and hence, other berm widths of 0.40 m and 0.45 m are not investigated.

NA[2] – Not applicable as the berm breakwater model with 0.35 m berm width and two layers of primary armor was not safe and hence, smaller berm width of 0.30 m was not investigated.

NA[3] – Not applicable as these berm breakwaters failed for berm widths of 0.45 m. Hence, shorter berm widths of 0.4 m, 0.35 m and 0.30 m were not investigated.

F indicates berm breakwater failure and S indicates safe berm breakwater structure.

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 SUMMARY

The physical model studies on cube armored conventional and reshaped berm breakwaters were conducted using wave flume in Marine Structures Laboratory of Applied Mechanics Department, National Institute of Technology Karanataka, Surathkal, India. In the first phase, cube armored conventional breakwater of 1V:1.5H sloped trapezoidal cross section was tested for non-breaking waves. In the second phase berm breakwater models were tested with cube armor with a slope of 1V:1.5H. The models were tested for stability by varying structural and sea state parameters. The aim of the study was to arrive at the design of an optimum configuration of statically stable reshaped berm breakwater with reduced concrete cube amour weight and required berm width. The conclusions drawn based on the present experimental work are listed in this chapter.

5.2 CONCLUSIONS FOR THE CONVENTIONAL BREAKWATER

Based on the experimental results of model study on conventional breakwater model constructed with concrete cube armor, the following conclusions are drawn.

1. Both the relative run-up (R_u/H_o) and the relative run-down (R_d/H_o) decrease with an increase in deepwater wave steepness (H_o/gT^2) and depth parameter (d/gT^2).
2. The damage level (S) increases with increase in H_o/gT^2, d/gT^2 and N_s.
3. Considering the complete ranges of H_o/gT^2 (i.e., $1.43 \times 10^{-3} \leq H_o/gT^2 \leq 7.78 \times 10^{-3}$) and d/gT^2 ($0.006 \leq d/gT^2 \leq 0.014$), the maximum relative run-up R_u/H_o and relative run-down R_d/H_o are respectively 1.2 and 1.25.
4. The breakwater damages are in the range of 4.62 to 5.69 (intermediate), 9.75 to 11.46 (failure) and 9.46 to 10.22 (failure) in the depths of 0.3m, 0.35m and 0.4m respectively.
5. Considering all the ranges of d/gT^2 (i.e. waves in all depths of water i.e., 0.3m, 0.35m and 0.4m), the increase in damage levels are 9.75 to 11.46 (17.5%), and 9.46 to 10.22 (8%) for waves of periods of 1.6 s. (i.e., $4.7 \times 10^{-3} \leq H_o/gT^2 \leq$

$7.85x10^{-3}$ and 2.0s ($2.5x10^{-3} \leq H_o/gT^2 \leq 4.0x10^{-3}$) respectively, and damage are zero for 2.6 s ($1.1x10^{-3} \leq H_o/gT^2 \leq 3.9x10^{-3}$).

6. Considering the waves in water depth of 0.35 m ($0.005 \leq d/gT^2 \leq 0.016$), the maximum damage levels increase from 9.46 to 9.76 (i.e. 10.66 %) and for waves in 0.40 m water depth i.e., $0.006 \leq d/gT^2 \leq 0.019$ the maximum damage increases from 10.22 to 11.46 (i.e., 9.78 %).

5.3 CONCLUSIONS FOR THE RESHAPED BERM BREAKWATER

Based on the experimental results of model study on statically stable reshaped berm breakwater, the following conclusions are drawn.

1. Berm recession, wave run-up, run-down and exposure of secondary layer are influenced by armor weight, berm width, wave height, period and depth, and storm duration.
2. The structure with reduced armor weight by 25% and 0.45 m berm width is safe for all the parameters considered in the present investigation.
3. For a model with 25% reduced armor weight and 0.40 m berm, increase in thickness from 2 layers to 3 layers of primary armor brought about 15% and 4% reduction in wave run-up and run-down respectively.
4. Breakwater model with 25% reduced armor weight, 0.40 m berm and 2 layers of primary armor is totally safe for almost all the test conditions except for extreme waves of 0.16 m height and 1.6 s period.
5. As relative berm position (h_b/d) parameter increases from 1.00 to 1.50, the berm recession decreased by up to 77%. Also, the wave run-up and run-down decreases by 7% and 14% respectively.
6. In stone armored berm breakwater, the berm recession is higher by 52.9%, compared to cube armored berm breakwater, and the wave run-up and run-down also more by 20.1% and 7.5% respectively for the same wave conditions.
7. A reduction upto 50.6% and 16.5% in wave run-up and run-down respectively can be achieved with a cube armored berm breakwater compared to conventional cube armored breakwater for the same wave conditions.

8. The present cube armored berm breakwater is 8% and 4% economical than the conventional cube armored breakwater and stone armored berm breakwater.
9. The berm recession equations available in the literature (Eqs. 2.35, 2.36, 2.39, 2.41 and 2.42 and 2.43) over estimate the R_{ec}/D_{n50} values by up to 83% compared to the one obtained in the present study.
10. The wave run-up equation given by CEM 2006 also over estimates the wave run-up compared to the present study.
11. The berm recession equation for statically stable reshaped berm breakwater with concrete cube as armor is derived as:

$$\frac{R_{ec}/D_{n50}}{N^{0.2}} = N_s \left[\frac{\left(H_o/gT^2\right)*10^{-3}}{n*\left(B/d\right)*\left(h_b/d\right)} \right]^{0.1}$$

12. The wave run-up and run-down equations based on the present experimental data are derived as:

$$\frac{R_u}{H_o} = \frac{2.449\xi_{eq}}{1+1.681\xi_{eq}} \text{ and}$$

$$\frac{R_d}{H_o} = \frac{1.680\xi_{eq}}{1+1.133\xi_{eq}}$$

5.4 SCOPE FOR FUTURE WORK

The following studies may be undertaken on the reshaped breakwaters:

1. Studies on the same model with random waves.
2. Varying structure slopes such as 1V:2H, 1V: 2.5H and 1V:3H with higher reduction in armor weight to get an optimum cross-section of the berm breakwater.

APPENDIX I

UNCERTAINTY ANALYSIS

AI-1 GENERAL

There are many ways of expressing the inaccuracies in an experiment. Error measurement is one of the method which measures of difference between true value and recorded value. An error is a fixed number and is not a statistical variable. Uncertainty analysis is another way of expressing an uncertainty which is different from error. An uncertainty is a possible value that the error might take on in a given measurement. Since, the uncertainty can take on various values over a range, it is inherently a statistical variable (Kline 1985). It is also generally agreed that the inaccuracies can be appropriately expressed by an "uncertainty" and these values could be obtained by an "Uncertainty analysis".

The hydrodynamic testing facilities differ from one another with regard to instrumentation, experimental procedures, scale, etc. Hence, it becomes necessary for a test facility to provide results with possible lower and upper margins, which can be accepted with a fair confidence level. The width of confidence intervals is a measure of the overall quality of the regression line. The 95% confidence interval limits must always be estimated and this concept of confidence level is fundamental to uncertainty analysis.

AI-2 CONFIDENCE AND PREDICTION BANDS

The plot of the best-fit curve can include the 95% confidence band of the best-fit curve, and the 95% prediction band. The two are very different. The Confidence band tells about the best fit curve. This means, it is 95% sure that the true best fit curve (if an infinite number of data points are available) lies within the confidence band. The prediction band tells about the scatter of the data. If data points are considered, 95% points are expected to fall within the prediction band. Since the prediction band has to an account for uncertainty in the curve itself as well as scatters around the curve, it is much wider than the confidence band. As increase in number of data points, the confidence band gets closer and closer to the best fit curve, while the prediction band doesn't change predictably.

For example, in Fig. AI-1 shown below, it is noticed that the confidence bands (shown as solid) contain a minority of data points. That indicates, the confidence bands have a 95% chance of containing the true best fit curve and with so much of data, these bands contain far fewer than half the data points. In contrast, the dashed prediction bands include 95% of the data points. 95% confidence and prediction bands have been accepted to be reliable enough for usage under the adoption of uncertainty analysis.

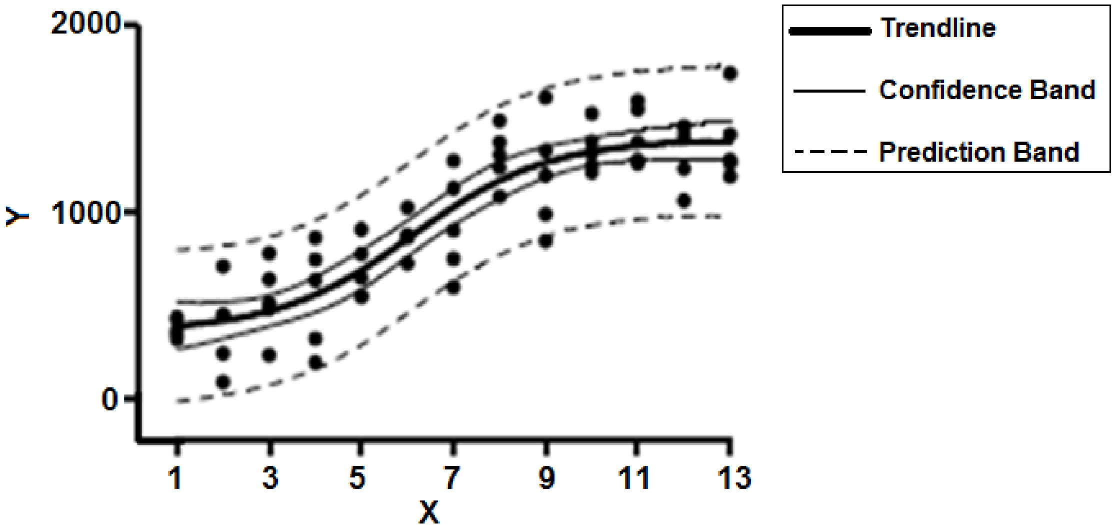


Fig. AI-1 Combined specimen graph for 95% confidence and prediction band

A 100(1-α) percent confidence interval about the mean response at the value of $x = x_1$, say Y_1 is given by Montgomery and Runger (1999):

$$Y_1 = Y_o \pm t_{(\alpha/2, n-2)} \sqrt{\left(\sigma^2 \left[\frac{1}{n} + \left(\frac{(x_o - \bar{x})^2}{S_{xx}}\right)\right]\right)} \text{ .... (AI-1)}$$

where, α = significance level used to compute the confidence level, σ^2 = variance, n = sample size, $\bar{x}$ = sample mean, x = variable, S_{xx} = standard deviation, $t_{(\alpha/2,\ n-2)}$ = t-distribution values for n – 2 degrees of freedom, and $Y_o = \beta_o + \beta_1 x_o$ is computed from the fitted regression model.

The prediction is independent of the observations used to develop the regression model. Therefore, the confidence interval for Y_o in Eq. AI-1 is inappropriate, since it is based only on the data used to fit the regression model. The confidence interval about Y_o refers to the true mean response at $x = x_o$, not to future observations.

A 100(1-α) percent prediction interval on a feature observation Y_o at given value x_o is given by:

$$Y_1 = Y_o \pm t_{(\alpha/2,n-2)} \sqrt{\left(\sigma^2 \left[1 + \frac{1}{n} + \left(\frac{(x_o - \bar{x})^2}{S_{xx}}\right)\right]\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(AI-2)}$$

where $Y_o = \beta_o + \beta_1 x_o$ is computed from the regression model.

AI-3 CONFIDENCE INTERVAL AND PREDICTION INTERVAL FOR DIMENSIONLESS BERM RECESSION WITH STABILITY NUMBER

The 95% confidence and prediction band for variation of recession level with stability number for reshaped berm breakwater models tested with W_{50} = 79.5 g, B = 0.45 m, T = 1.6 to 2.6 s, H = 0.10 to 0.16 m and d = 0.30 to 0.45 m is shown in Fig. AI-2. It can be observed that more than 90% of experimental data lie within the 95% confidence bands. The coefficient of determination, R^2, is found to be 0.92.

Fig. AI-2 Plot of 95% confidence and prediction bands for the variation of R_{ec}/D_{n50} with N_s

From the figure it is observed that the trend line lie within these 95% confidence bands and data points lie within the 95% prediction bands drawn. From the bands drawn, the results may be analyzed with 95% confidence i.e. the conclusions drawn from these graphs are 95% reliable. Also from the figure it may be visualized that experimental data points are bounded within the 95% prediction bands and this particular observation strengthen the conclusions derived from this graph.

AI-4 CONFIDENCE INTERVAL AND PREDICTION INTERVAL FOR RELATIVE RUN-UP WITH SURF SIMILARITY PARAMATER

The 95% confidence interval and prediction interval for variation of relative run-up with surf similarity parameter for reshaped berm breakwater models tested with W_{50} = 79.5 g, B = 0.45 m, T = 1.6 to 2.6 s, H = 0.10 to 0.16 m and d = 0.30 to 0.45 m are shown in Fig. AI-3. It has been observed that around 95% of the experimental data lie within the 95% confidence bands. The coefficient of determination, R^2, is found to be 0.895.

Fig. AI-3 Plot of 95% confidence and prediction bands for the variation of R_u/H_o with ξ_{eq}

AI-5 CONFIDENCE INTERVAL AND PREDICTION INTERVAL FOR RELATIVE WAVE RUN-DOWN WITH SURF SIMILARITY PARAMETER

The 95% confidence interval and prediction interval for variation of relative run-down with surf similarity parameter for reshaped berm breakwater models tested with W_{50} = 79.5 g, B = 0.45 m, T = 1.6 to 2.6 s, H = 0.10 to 0.16 m and d = 0.30 to 0.45 m are shown in Fig. AI-4. The coefficient of determination, R^2, is found to be 0.895.

Fig. AI-4 Plot of 95% confidence and prediction bands for the variation of R_d/H_o with ξ_{eq}

As it can be observed about 70% data points lie within the confidence band and all the points are within prediction band. The large scatter is because of different wave conditions and water depths considered during the study. Also, the wave run-down includes both down rush of a wave and flow from within the pores of the armor layer of the breakwater. This combined flow increases the variation in the run-down for considered parameters.

AI-6 SPECIMEN CALCULATION OF CONFIDENCE AND PREDICTION INTERVALS

Specimen calculation for the plot R_u/H_o vs. ξ_{eq} for different depths of water is shown in Table AI-1 with the 95% upper and lower confidence band values and 95% upper and lower prediction band values. The values are computed using Eqs. AI-1 and AI-2.

Table AI-1 Data points with results of 95% confidence band and 95% prediction band

X_i (ξ_{eq})	Y_i (R_u/H_o)	Confidence band		Prediction band	
		LL Confidence	UL Prediction	LL Confidence	UL Prediction

1.628	1.021	1.018	0.964	1.127	0.855
1.487	1.002	1.000	0.938	1.107	0.832
1.376	0.981	0.987	0.918	1.090	0.814
1.287	0.954	0.977	0.901	1.078	0.800
2.083	1.072	1.084	1.038	1.197	0.925
1.901	1.066	1.056	1.011	1.169	0.898
1.760	1.047	1.035	0.987	1.147	0.876
1.647	1.038	1.020	0.968	1.130	0.858
2.823	1.195	1.221	1.129	1.316	1.034
2.577	1.152	1.174	1.100	1.276	0.999
2.386	1.139	1.138	1.078	1.245	0.971
2.232	1.118	1.110	1.058	1.220	0.948
1.636	1.007	1.019	0.966	1.129	0.856
1.493	0.967	1.001	0.940	1.108	0.833
1.383	0.947	0.988	0.919	1.091	0.815
1.293	0.924	0.977	0.902	1.078	0.801
2.100	1.067	1.087	1.040	1.199	0.928
1.917	1.043	1.058	1.013	1.171	0.900
1.775	1.026	1.037	0.990	1.149	0.878
1.660	1.010	1.022	0.970	1.132	0.860
2.855	1.175	1.227	1.132	1.322	1.038
2.606	1.141	1.179	1.104	1.280	1.003
2.413	1.122	1.143	1.081	1.249	0.975
2.257	1.091	1.114	1.061	1.224	0.952
1.647	0.960	1.020	0.968	1.130	0.858
1.503	0.941	1.002	0.941	1.109	0.835
1.392	0.928	0.989	0.921	1.093	0.817
1.302	0.897	0.978	0.903	1.080	0.802
2.122	1.038	1.091	1.043	1.203	0.931
1.937	0.998	1.061	1.016	1.174	0.903
1.793	0.987	1.040	0.993	1.152	0.881

1.677	0.962	1.024	0.973	1.135	0.863
2.891	1.157	1.234	1.136	1.328	1.043
2.639	1.121	1.186	1.108	1.286	1.007
2.443	1.094	1.148	1.085	1.254	0.979
2.285	1.072	1.119	1.065	1.229	0.956

APPENDIX II

COST ANALYSIS

AII-1 INTRODUCTION

From the model studies on berm breakwater, it is concluded that the geometry of the berm breakwater with 25% reduced armor weight with a berm width of 0.45 m is completely safe. Considering this, the trunk section of a conventional and a berm breakwater armored with cube units are designed for the same operating conditions. This chapter explains the details of the analysis and comparison of cost of prototypes of conventional and berm breakwater.

AII-2 DESIGN OF CONVENTIONAL BREAKWATER

AII-2.1 Armor weight

Based on the investigations conducted in the wave flume, the trunk section of a conventional non-overtopping breakwater of primary armor weight (W) of 2.9 T is designed using Hudson formula (1959) with cubes as armor unit. The weight of stones in the secondary layer is W/10 = 300 kg but stones of 200 kg to 500 kg are used. A toe of 200 kg to 500 kg stones, of top width 4.8 m and 3.6 m high is provided at the bottom at both the ends. The weight of core material is W/200 = 15 kg but 10 kg to 100 kg are used.

AII-2.2 Breakwater height

The maximum run-up observed is about 1.75 times the wave height, about 3.0 m, in a water depth of 12 m. This results in a breakwater height of 17.3 m. However, breakwater crest elevation is fixed at 21 m.

AII-2.3 Thickness of layers

The thickness of primary and secondary layer is calculated as 3.7 m and 1.6 m respectively. The minimum crest width should be sufficient to accommodate 3 armor units and crest width of 3.5 m is provided.

AII-2.4 Design parameters of conventional breakwater

Design wave height, H		: 3.0 m
Design armor weight, W		: 2.9 T
Specific weight of armor, S_r		: 2.4
Mass density of armor unit		: 2.4 T/cum
Relative mass density of armor unit, ρ_a		: 1.4 T/cum
Layer coefficient, K_Δ		: 1.1
Porosity of	Primary layer	: 47%
	Secondary layer	: 39%
	Core	: 36%
Breakwater slope		: 1V : 1.5H
Thickness of	Primary layer	: 3.5 m
	Secondary layer	: 1.7 m
Depth of water		: 12 m
Breakwater height		: 21 m
Crest Width		: 3.5 m
Weight of	Secondary layer, W/10	: 200 kg to 500 kg
	core, W/200	: 10 kg to 100 kg
	Toe, W/10	: 200 kg to 500 kg

Fig. AII-1 gives the dimensions of the conventional breakwater designed.

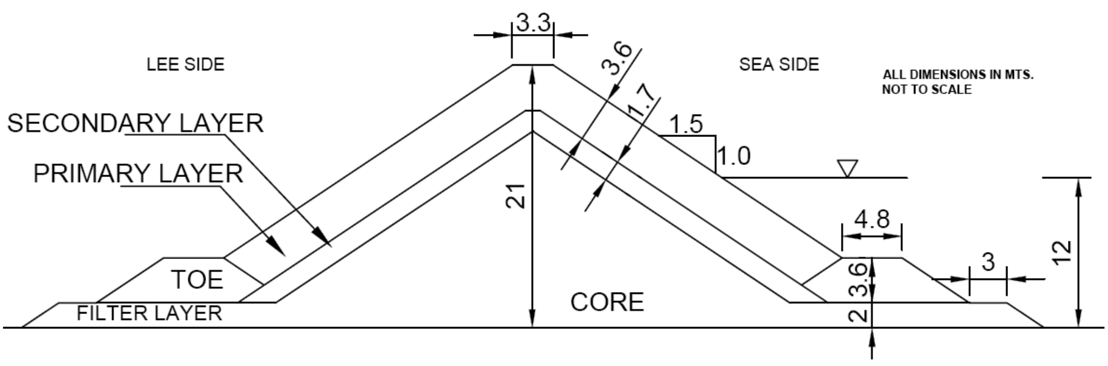


Fig. AII-1 Conventional breakwater section

AII-3 DESIGN OF BERM BREAKWATER

Based on the conclusions of the present study, it is decided to design the trunk section of a berm breakwater with cube of armor weight reduced by 25% of that of the conventional breakwater. Hence, the armor weight for berm breakwater is 2.2 T. The weight of stones in the secondary layer is W/10 = 230 kg but stones of 200 kg to 500 kg are used. A toe of 200 kg to 500 kg stones, of top width 4.8 m and 3.6 m high is provided at the bottom at both the ends. The weight of core material is W/200 = 12 kg but 10 kg to 100 kg are used.

The maximum run-up in case of berm breakwater is 1.2 times the wave height in a water depth of 12 m which will result in breakwater height of 15.6 m. However, breakwater crest elevation is fixed at 18 m. A berm of width 12 m will be provided at an elevation of 13.5 m from the sea bed. The thickness of primary and secondary layer is calculated as 2.2 m and 1.0 m respectively. The minimum crest width should be sufficient to accommodate 3 armor units and crest width of 3 m is provided.

AII-3.1 Design parameters of berm breakwater

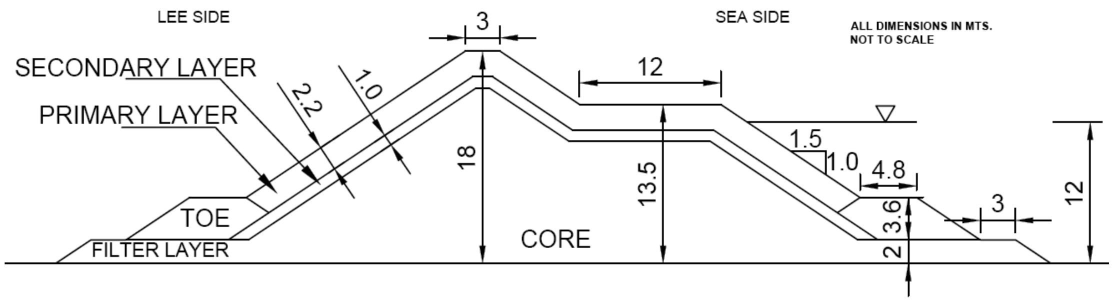
Design wave height, H		: 3.0 m
Design armor weight, W		: 2.2 T
Specific weight of armor, S_r		: 2.4
Mass density of armor unit		: 2.4 T/cum
Relative mass density of armor unit, ρ_a		: 1.4 T/cum
Layer coefficient, K_Δ		: 1.1
Porosity of	Primary layer	: 47%
	Secondary layer	: 39%
	Core	: 36%
Breakwater slope		: 1V : 1.5H
Berm width, B		: 12 m
Position of berm from sea bed, h_B		: 13.5 m
Thickness of	Primary layer	: 2.15 m
	Secondary layer	: 1.0 m
Depth of water, d		: 12 m

Breakwater height : 18 m

Crest Width : 3 m

	Secondary layer, W/10	: 200 kg to 500 kg
Weight of	core, W/200	: 10 kg to 100 kg
	Toe, W/10	: 200 kg to 500 kg

Fig. AII-2 gives the dimensions of the berm breakwater designed.



AII-2 Berm breakwater Section

AII-4 QUANTITY ESTIMATION

The density of cube armor for the calculation of their quantity is estimated as follows:

AII-4.1 Sample calculation

Porosity of primary armor layer = 0.47

Taking volume of primary layer = 1 m^3

Volume of voids, V_v = 0.47 m^3

Volume of cubes in the layer = 0.53 m^3

Weight of cubes in 1 m^3 of the layer = volume of cubes X mass density of cubes

= 0.53 X 2.4

= 1.3 T

Density of primary layer = 1.3 T/m^3

Similarly,

Density of secondary layer = 1.7 T/m^3

Density of core = 1.8 T/m^3

AII-4.1 Quantity Estimation

The quantity of cube armors required for conventional and berm breakwaters are calculated in this section. The quantity of material of the breakwater section is first calculated and these are increased by 15% for possible settlement and/or soft clay conditions due to its large cross section and a heavy load on the foundation.

Table AII-1 gives the estimation of quantity of primary armor, secondary armor and core as per the design parameters given in section AII-2.4 and with reference to Fig. AII-1 for conventional breakwater and Table AII-2 gives the same for berm breakwater as per the design parameters given in section AII-3.1 and with reference to Fig. AII-2. The area (BXD) is obtained by drawing the breakwater structures to prototype dimensions in software called "AutoCAD 2009". The total quantity in the tables indicates the increased quantity by 15% due to settlement and soft clay conditions.

Table AII-1 Quantity estimation of cube armored conventional breakwater for per meter length

Sl. No	Layer	Density (T/m^3)	No	Length (m)	Area (BXD) (m^2)	Volume (m^3)	Quantity (T)	Increased quantity (by15%) (T)
1	Filter Layer (1 to 5 kg)	1.8	2	1.0	28.70	57.40	103.320	118.818
2	Core Layer (5 to 100 kg)	1.8	1	1.0	371.16	371.16	668.088	768.301
3	Sec. Layer (200 to 500 kg)	1.7	1	1.0	90.21	90.21	153.357	176.361
4	Primary Layer (2.9 T)	1.3	1	1.0	197.12	197.12	256.256	294.694
5	Toe Layer (200 to 500 kg)	1.7	2	1.0	33.55	67.10	114.07	131.181

Table AII-2 Quantity estimation of stone armored berm breakwater for per meter length

Sl. No	Layer	Density (T/m^3)	No	Length (m)	Area (BXD) (m^2)	Volume (m^3)	Quantity (T)	Increased quantity (by15%) (T)
1	Filter Layer (1 to 5 kg)	1.8	2	1.0	20.77	41.54	74.772	85.988
2	Core Layer (5 to 100 kg)	1.8	1	1.0	474.55	474.55	854.190	982.318
3	Sec. Layer (200 to 500 kg)	1.7	1	1.0	58.55	58.55	99.535	114.465
4	Primary Layer (2.9 T)	1.6	1	1.0	123.65	123.65	197.84	227.516
5	Toe Layer (200 to 500 kg)	1.7	2	1.0	28.73	57.46	97.682	112.334

Table AII-3 Quantity estimation of cube armored berm breakwater for per meter length

Sl. No	Layer	Density (T/m^3)	No	Length (m)	Area (BXD) (m^2)	Volume (m^3)	Quantity (T)	Increased quantity (by15%) (T)
1	Filter Layer (1 to 5 kg)	1.8	2	1.0	20.77	41.54	74.772	85.988
2	Core Layer (5 to 100 kg)	1.8	1	1.0	474.55	474.55	854.190	982.318
3	Sec. Layer (200 to 500 kg)	1.7	1	1.0	58.55	58.55	99.535	114.465
4	Primary Layer (2.9 T)	1.3	1	1.0	123.65	123.65	160.745	184.856
5	Toe Layer (200 to 500 kg)	1.7	2	1.0	28.73	57.46	97.682	112.334

AII-5 ANALYSIS OF RATES

Unit rates of all items of work are calculated for 10 m^3 of material which includes construction, quarrying, blasting of hard rock, depositing and stacking of useful stones within a lead of 50 m, transportation, weighing and dumping/placing at specified location with a lead of 8 km from the quarry as shown in Table AII-4. Finally cost per ton of armor stone is derived. The cost of 1 m^3 of concrete (M35 grade) is Rs. 6500/- which gives cost per ton equal to Rs. 1,965/-. The cost of preliminaries like equipments, mobilization, insurance, temporary construction of temporary jetties etc., are not included but are to be estimated separately and added to the cost of breakwaters. Table AII-5 gives the final unit rates of armor stones of various sizes.

Table AII-4 Calculation of unit rates of civil works

Sl. No.	Description of work	Quantity	Rate (Rs.)	Per	Amount (Rs.)
1	**Materials**				
	Country blasting powder	4 kg	380/-	1 kg	1,520.00
	Country fuse	12 m	200/-	1 m	2,400.00
2	**Labor**				
	Quarry men(for boring holes)	15 Nos.	500/-	each	7,500.00
	Quarry men(charging holes with Powder and tampering and firing)	2 Nos.	500/-	each	1,000.00
	Hammer men(Breaking big boulders)	2 Nos.	350/-	each	700.00
	Men (Removing blasted rock to a distance of 50 mts)	3 Nos.	350/-	each	1,050.00
3	Loading, Unloading and Transporting charge (8 kms).				2,000.00
					16,170.00
4	Royalties @ 10% on Rs 5,097/-				1,617.00
5	Weighing charge Lump sum				275.00
6	Carriage charge of stone up to breakwaters site and placing				
	Hire charge of boat or any other transportation upto breakwater construction site into the sea.				1,200.00
	Loading and placing in position the stones with necessary lifting equipment.				900.00

7	Sundries, wastage – 10%				2,016.00
8	Add contractor’s overhead and profit 20%.				4,436.00
					26,614.00
9	So, cost per cubic meter (for average size & weight)	$\frac{26,614}{10}$			**2,660.00**
10	Cost per Ton	$\frac{(2660 \times 1,000)}{2800}$			**950.00**

Table AII-5 Final unit rates of armor stones of various sizes

Sl. No.	Description of layer	Rate per Ton (Rs.)
1	Filter layer and core stones (1kg to 5kg)	950.00
2	Core and Secondary layer stones (5kg to 100kg) (Add 5% extra over ‘1’)	998.00
3	Secondary layer stones (100kg to 200kg) (Add 5% extra over ‘2’)	1,048.00
4	Primary layer stones (200kg to 500kg) (Add 5% extra over ‘3’)	1,100.00
5	Primary layer stones (500kg to 700kg) (Add 5% extra over ‘4’)	1,155.00
6	Primary layer stones (700kg to 1000kg) (Add 5% extra over ‘5’)	1,213.00
7	Primary layer stones (1000kg to 2000kg) (Add 25% extra over ‘6’)	1,517.00
8	Primary layer stones (2000kg to 3000kg) (Add 25% extra over ‘7’)	1,900.00

AII-6 COST OF CONSTRUCTION

The cost of construction of conventional breakwater is shown in Table AII-6. This is calculated considering the estimation of armor and other material quantities given in Table AII-1 and the final unit rates calculated in Table AII-5. The cost of construction of berm breakwater armored with stone is shown in Table AII-7. This is calculated considering the estimation of armor and other material quantities given in Table AII-2 and the final unit rates calculated in Table AII-5. Similarly, the cost of construction of berm breakwater armored with cube is shown in Table AII-8. This is calculated considering the estimation of armor and other materials given in Table AII-2 and the unit rates given in Section AII-5.

Table AII-6 Cost of construction of cube armored conventional breakwater

Sl. No.	Description of work	Quantity Ton	Rate Rs/Ton.	Amount Rs.
a)	Filter layer (1kg to 5kg)	118.818	950	1,12,877
b)	Core stones (10kg to 100kg)	768.301	998	7,66,765
c)	Secondary layer (200kg to 500kg)	176.361	1,100	1,93,997
d)	Primary layer (2.9T) (Cube)	294.694	1,965	5,79,074
e)	Toe berms (200kg to 500kg)	131.181	1,100	1,44,299
	TOTAL COST (Rs.)			**17,97,012**

Table AII-7 Cost of construction of stone armored berm breakwater

Sl. No.	Description of work	Quantity Ton	Rate Rs/Ton.	Amount Rs.
a)	Filter layer (1kg to 5kg)	85.988	950	81,689
b)	Core stones (10kg to 100kg)	982.318	998	9,80,354
c)	Secondary layer (200kg to 500kg)	114.465	1,100	1,25,912
d)	Primary layer (2T to 3T)	227.516	1,900	4,32,281
e)	Toe berms (200kg to 500kg)	112.334	1,100	1,23,568
	TOTAL COST (Rs.)			**17,43,804**

Table AII-8 Cost of construction of cube armored berm breakwater

Sl. No.	Description of work	Quantity Ton	Rate Rs/Ton.	Amount Rs.
a)	Filter layer (1kg to 5kg)	85.988	950	81,689
b)	Core stones (10kg to 100kg)	982.318	998	9,80,354
c)	Secondary layer (200kg to 500kg)	114.465	1,100	1,25,912
d)	Primary layer (2.2T) (Cube)	184.856	1,965	3,63,242
e)	Toe berms (200kg to 500kg)	112.334	1,100	1,23,568
	TOTAL COST (Rs.)			**16,74,765**

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